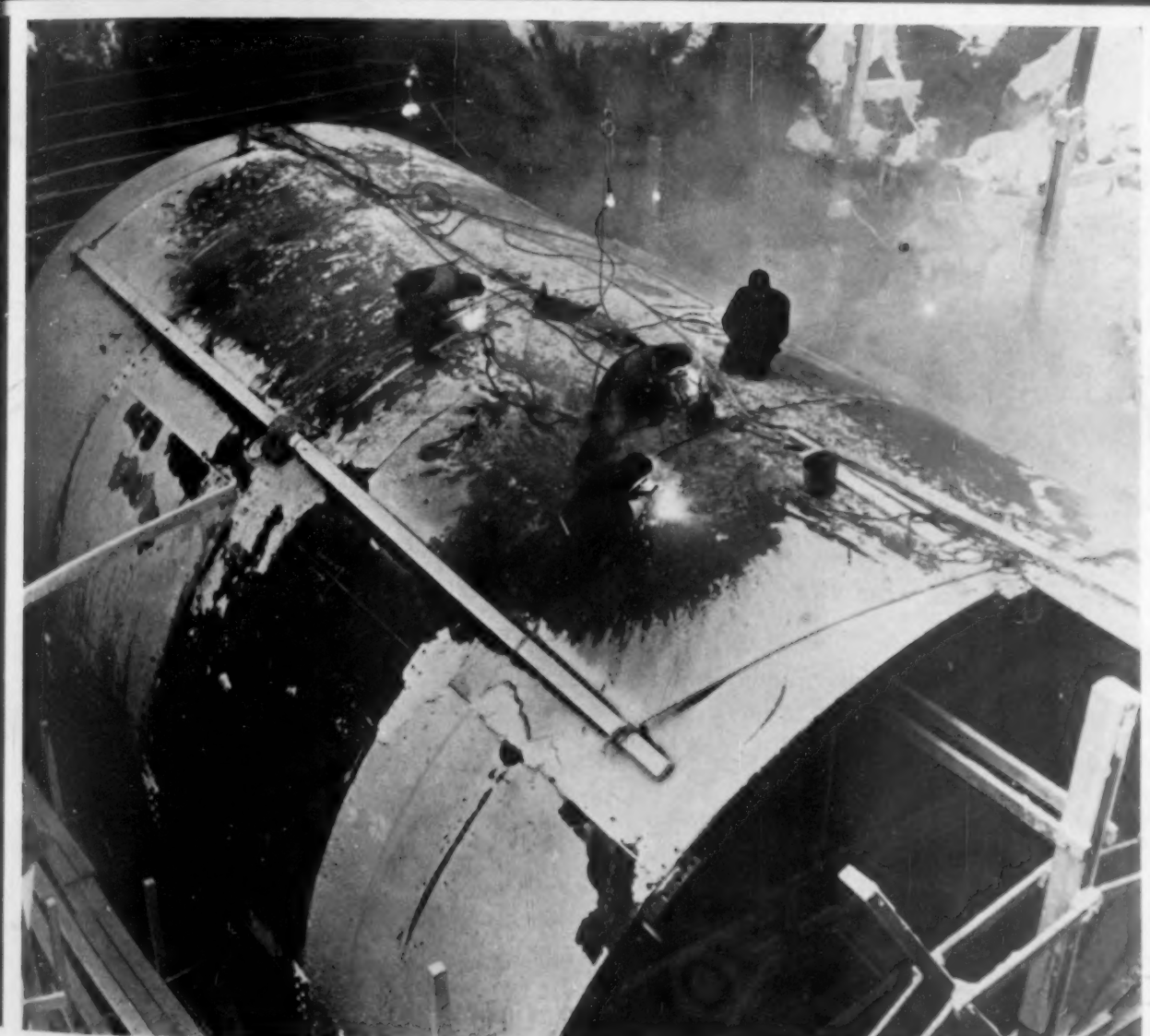
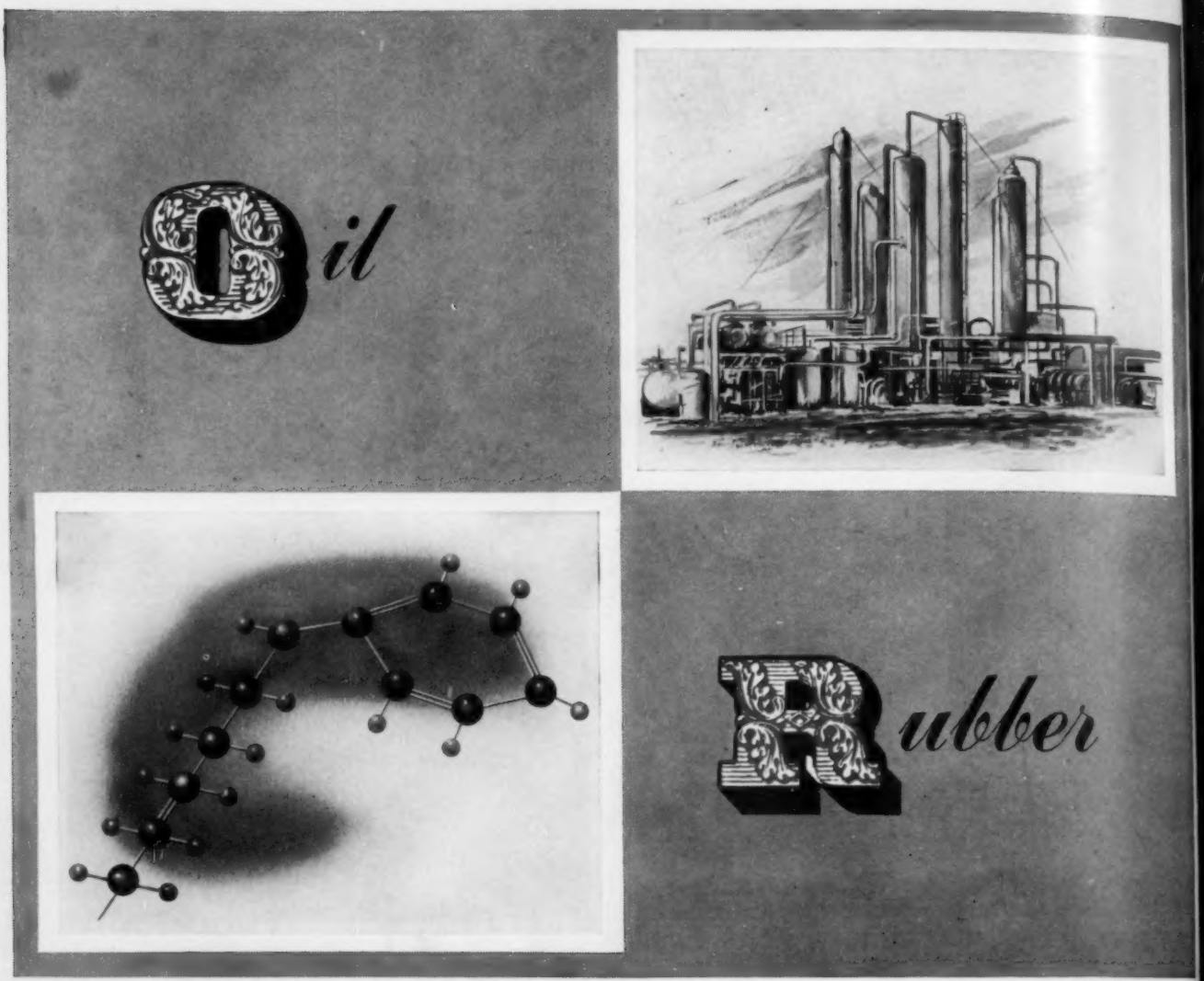
SEPTEMBER 1943

WELDING Journal



The Journal of the American Welding Society



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The Journal of the American Welding Society

Published for the Advancement of the Science and Art of Welding

W. SPRARAGEN, Editor

September 1943

VOLUME 22 NUMBER 9

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Welding Operators at Work in Snow on Casing to Line One of Six 30-foot Penstocks for Water at Shipshaw, Canada. *(Photo by National Film Board.) Courtesy General Electric Company.*

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The Practical Design of Welded Steel Structures*

By H. Malcolm Priest†

EDITORIAL NOTE: Ever since the publication of the first paper by H. M. Priest, on "The Practical Design of Welded Steel Structures," in the August 1933 issue of THE AMERICAN WELDING SOCIETY JOURNAL, which dealt largely with bare wire welding, there has been an insistent demand by members of the SOCIETY that Mr. Priest be requested to bring his paper up to date and revise tables and figures on the basis of the use of modern shielded arc electrodes. The SOCIETY is deeply grateful to the author for complying with its request and the present paper represents more than a year of effort on his part.

Introduction

TEN years have elapsed since the writer presented a paper with this same title before the New York Section of the AMERICAN WELDING SOCIETY. Then, welding was emerging from the earlier period of uncertainties, bare electrodes were in common use and the future still held the notable advances in both the technical and practical phases of the art that are now common knowledge. Today, there is a better understanding and a clearer comprehension of the natural characteristics of proper welding design, the simplicity of which operated to obscure its philosophy.

During these ten years, research has been broad and continuous, and the fields of application have been steadily expanding. Manufacturers have maintained a constant progress in the development of welding equipment and in the introduction of electrodes to meet the diverse needs encountered in the growing industrial use of welding.

All of these developments have had a definite influence upon the design of welded structures. The superior quality of welds made with coated electrodes has been recognized in the adoption of higher working unit stresses for such welds. Butt welds have come into increasing favor, due to the more general recognition of their greater freedom from stress concentrations and consequent greater strength in fatigue. In some cases, butt welds also show a lower cost of production. Much study has also been given to the problem of properly utilizing the inherent rigidity of welded connections to obtain an economy of material. This study has found expression in designs embodying the principles of continuity.

It seemed, at first, a simple undertaking to make the requested revision of the original paper, but the final result has been the rewriting of large portions and the addition of considerable new material. Only by constantly guarding against expansion has it been possible to keep the material within the bounds of an already lengthy paper. The fundamentals of design remain the same, but the details of execution undergo improvement with the guidance of experience. Unchanged is the aim of the original paper, namely, "to present the essentials of the design of welded steel construction, with a sufficient background of collateral information that the reader may have a somewhat clearer understanding of structural welding" and its current status.

It no longer seems necessary to list the sources of welding information, but one source does merit particular mention. Concurrent with the development of welding has been the growth of the AMERICAN WELDING SOCIETY. Through the work of this SOCIETY and its numerous technical committees, its Codes and Specifications have received general recognition and their authority and influence are widely felt. No designing engineer should be without these standards of construction practice. THE WELDING JOURNAL of the SOCIETY and the publications of the Welding Research Committee* of the Engineering Foundation carry many significant contributions to the literature of welding. The important codes and specifications are listed in the bibliography at the end of this paper.

Welding Processes

The approved definition of a weld is "a localized consolidation of metals by means of a welding process." It is a common characteristic of all processes that heat is generated and required to bring the metals to a plastic or molten state. Figure 1 is the Master Chart of Welding Processes classifying the principal processes. Most structural welding is done by the electric arc (fusion) process. One of the pressure processes, namely, electric resistance welding is extensively used in the manufacture of steel joists and light trusses of skeleton form. It is not within the scope of this article to discuss at length the technique of the welding operations, but rather to present only a general picture which will add to an intelligent appreciation of the subsequent remarks.

Metal Arc Welding

The welding heat is obtained from the electric arc formed between the electrode and the parts to be welded.

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† Engineer, Railroad Research Bureau, U. S. Steel Corporation Subsidiaries; formerly Designing Engineer, American Bridge Company.

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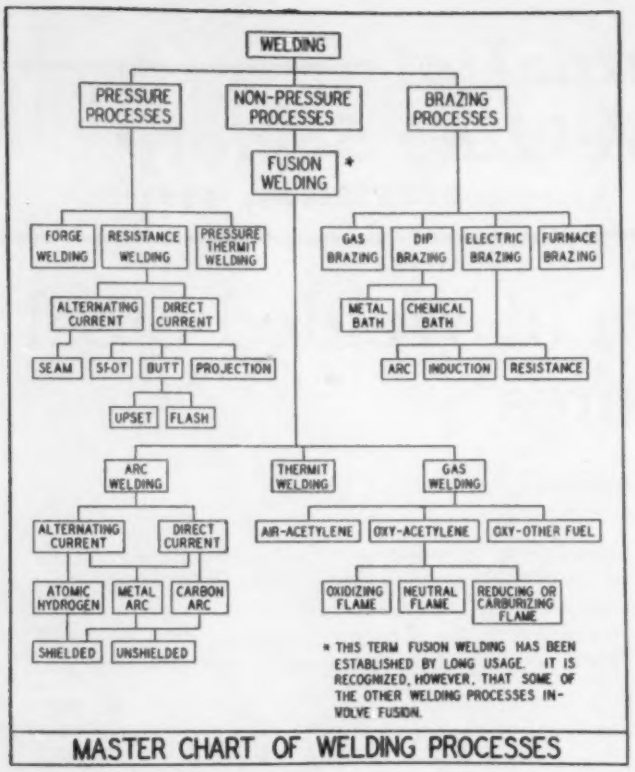

Fig. 1

Both direct and alternating currents are employed. Figure 2 (*A*) represents the direct-current process in which a generator, driven by electric motor or gasoline engine, supplies the current. Investigators have found that more heat is liberated on the positive side of the arc than on the negative; hence the usual practice of attaching the positive terminal of the generator to the base metal (parts to be welded) which is of greater extent than the electrode wire and requires more heat to bring it up to the welding temperature. Under certain conditions, as when welding thin material or using some of the coated electrodes, the connections are reversed, giving rise to the expression, "welding with reversed polarity."

A schematic diagram for the alternating-current process would be the same as shown in Fig. 2 (*A*), except that with an a.-c. generator the terminals are alternately positive and negative. Where a source of alternating-current power is available it is possible to use transformers to convert line voltages to those employed in welding and so eliminate the necessity for a motor or engine to drive an a.-c. generator.

In the process of arc welding, the electrode melts and unites with the base metal, which also melts in a localized or relatively small area, to form the weld. A distinctive feature of arc welds is the crater formed directly under the electrodes. It is common practice to fill this crater at the end of a weld by melting sufficient electrode to do so.

Oxyacetylene (Gas) Welding

Figure 2 (*B*) depicts the essentials of the gas-welding process in which the welding heat is derived from the combustion of acetylene and oxygen at the tip of a blowpipe or torch. The torch can be fitted with interchangeable tips with varying sizes of orifices, depending upon the amount of heat required. Suitable gages and regulators permit of the control of the gas pressures to obtain the best results. A localized area of the base metal is brought up to the proper welding temperature by directing the flame upon it, after which the welding rod is introduced into the flame and is melted to form the weld.

Carbon Arc Welding

Carbon arc welding, Fig. 2 (*C*), is similar to gas welding, in that a welding rod is introduced into the arc formed between the carbon electrode and the base metal. In some cases, when welding thin materials, the heat of the arc is used to melt the parts and fuse them together without the addition of any filler material from a welding rod.

Resistance Welding

The resistance welding process is one of great flexibility and takes many different forms, depending upon the particular application. The principle of resistance welding, Fig. 2 (*D*), consists in passing an alternating current of high amperage and low voltage through metals, with pressure applied at some stage of the process, and using the heat produced by the electrical resistance at the point of contact to bring the material to a plastic state, whereupon welding is accomplished by continued or additional pressure.

In spot welding, the parts to be welded are placed between two electrodes, pressure is applied through the electrodes and the current fuses the base metal parts in the region between the electrode tips. Many commercial welding machines are equipped with a series or bank of electrodes which enable the operator to make a large number of spot welds with one placement of the material.

Two common forms of resistance butt welding are known as "push butt welding" and "flash butt welding." In the former, the ends or edges of the parts to be welded are held in clamps and brought into alignment and contact under moderate pressure. The passage of current through the joint brings the edges to a plastic state when additional pressure is applied, which upsets the joint and produces a fusion of the parts in a manner similar to forge welding. Flash butt welding is carried out by bringing the edges into light contact and during this

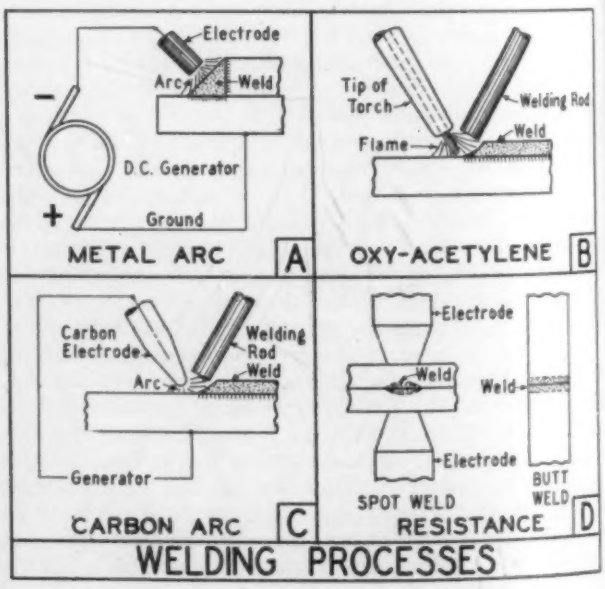

Fig. 2

stage the current arcs across the scattered points of contact, which increase rapidly in number until the entire lengths of the edges are brought to a welding temperature, when pressure is suddenly applied, upsetting the plastic edges and thereby forming the weld.

Submerged Melt Welding Process

This process, known as Unionmelt welding, is a fully automatic electric process in which the end of the electrode is constantly covered by a special granulated material or welding composition. Figure 3 depicts the process, showing how the granulated material is laid down along the seam to be welded. The welding action takes place beneath this layer of granulated material, during which time the bare metal electrode and a portion of the edge of the base metal are melted and fused. The granulated material performs several functions that make for clean, dense weld metal having excellent mechanical properties. The process is particularly advantageous for repetitive operations where welds can be made with the surface of the welding puddle horizontal. Because of the high currents that can be used, it is possible to make butt welds in one pass on thicknesses of material that would require many passes in manual welding. As examples of the high speed of welding attainable with this process, it may be cited that high-quality butt welds can be made at 10 to 12 in. per minute in 1-in. steel plates, and at 5 to 6 in. per minute in 2-in. plates.

Welding Rods and Electrodes

There are many rods and electrodes on the market from which the user may select the type best suited to his particular requirements. Electrodes are commonly divided into three classes:

1. Bare wire.
2. Lightly coated wire.
3. Heavily coated wire.

Most welding today is done with heavily coated (shielded arc) electrodes which have largely displaced the bare wire electrodes of earlier days. This change was brought about because of the superior characteristics of the weld metal, such as cleanliness, ductility and increased tensile strength, obtained with these heavily coated electrodes and the increased rate of deposition which they make possible. The coating serves to stabilize the arc and to form a protective slag over the weld that retards the rate of cooling of the weld metal and protects the hot metal from atmospheric gases. Another important feature is that the mechanical properties and chemical composition of the deposited weld metal can be controlled by variations in the chemical composition of the coating.

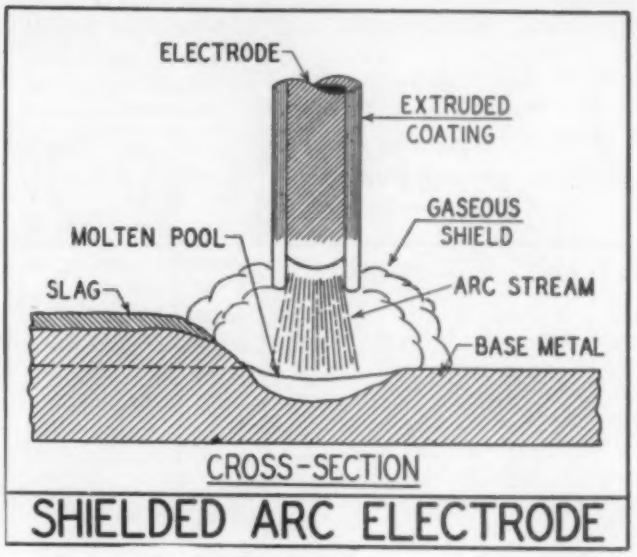

Fig. 4

Figure 4 is a cross section illustrating the essential characteristics of the shielded arc electrode welding process. It depicts the steel electrode with its extruded coating, the arc stream surrounded by the gaseous shield which also protects the molten pool, and then the protective layer of slag which floats to the top and quickly solidifies over the hot weld metal. When cool, the slag is readily removed by light taps from a hammer or chisel.

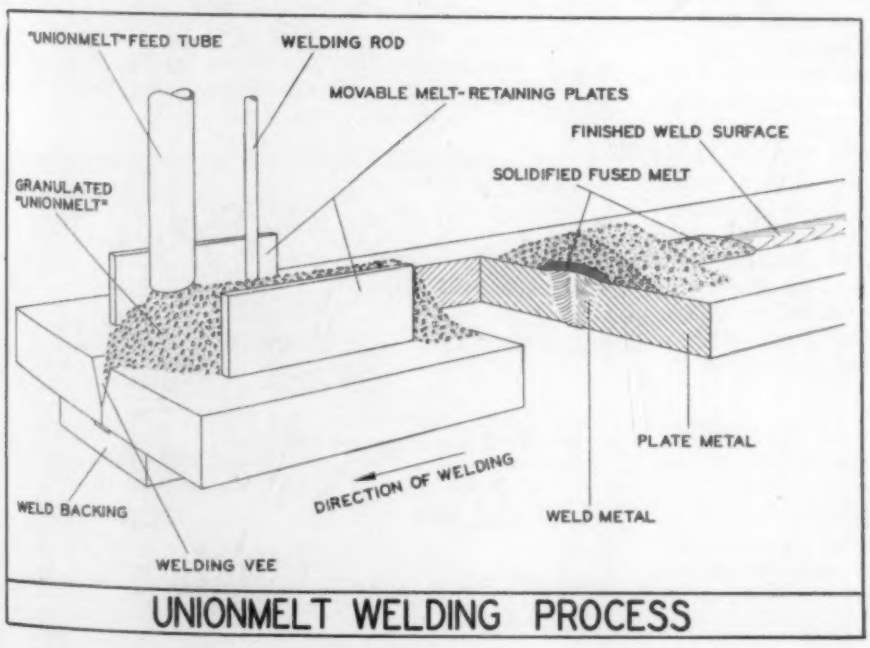

Fig. 3

Some electrodes are suited only to certain positions of welding or are best adapted to certain conditions of fit between the parts to be welded. Other types are designed for welding cast iron, high-tensile steels, or special alloys such as stainless steels. The fabricator has a wide choice in selecting an electrode for a specific purpose.

The manufacturers of electrodes quite generally state the voltage and current values to be used with their product. These vary with the size of wire and the thickness of the material to be welded. The smaller the wire or the thinner the material, the less the current should be for the proper results. Too heavy current is apt to burn the steel and to produce porosity in the welds. It is, therefore, wise to be guided by the advice of responsible manufacturers.

TENSILE REQUIREMENTS OF DEPOSITED METAL

Electrode Classification Number	Capable of Producing Satisfactory Welds in Positions Shown	Treatment of Welded Specimen	Tension Test Requirements of Material Deposited from 1/8 to 5/16 in. Electrodes		
			Tensile Strength min. psi.	Yield Point min. psi.	Elongation in 2 in. min. per cent
E 6010	F V OH H	SR	60 000	47 000	27
		NSR	62 000	52 000	22
E 6011	F V OH H	SR	60 000	47 000	27
		NSR	62 000	52 000	22
E 6012	F V OH H	SR	60 000	47 000	22
		NSR	62 000	52 000	17
E 6013	F V OH H	SR	60 000	47 000	22
		NSR	62 000	52 000	17
E 6020	H-Fillets F	SR	60 000	47 000	30
		NSR	62 000	52 000	25
E 6030	F	SR	60 000	47 000	30
		NSR	62 000	52 000	25

F = Flat, V = Vertical, OH = Overhead, H = Horizontal, SR = Stress-Relieved, NSR = Non-Stress-Relieved

SPECS. FOR ARC-WELDING ELECTRODES

Fig. 5

The AMERICAN WELDING SOCIETY and the American Society for Testing Materials have jointly issued two tentative specifications for arc-welding electrodes and gas-welding rods. These are:

(*a*) Tentative Specifications for Iron and Steel Arc-Welding Electrodes. Serial Designation: A233–42T.*

(*b*) Tentative Specifications for Iron and Steel Gas-Welding Rods. Serial Designation: A251–42T.*

These specifications cover coated electrodes and gas-welding rods for the welding of carbon and low-alloy steels of weldable quality. They are classified on the basis of usability and the ultimate tensile strength of all-weld-metal specimens in the stress-relieved condition.

Figure 5 is a tabulation of the tensile requirements of deposited metal of the electrodes commonly used in structural work. Each electrode classification is given a number, such as E6010, and in the second column of the table are listed the welding positions in which the electrodes may be used. The tensile test requirements are based upon all-weld-metal specimens in the as-welded condition, designated by the letters *NSR*, and in the stress-relieved condition, designated by *SR*. The reason for stress relieving a portion of the specimens is to secure information as to the inherent physical characteristics of the weld metal, apart from any locked-up stresses or heat effects that might be present in the as-welded condition. This does not imply that stress relieving is required for construction welded with these electrodes.

Electrode	Application
E6010	Useful with d.c., electrode positive only
E6011	Useful with a.c. only
E6012	Usually used with electrode negative, d.c. or a.c.
E6013	Usually used on a.c.
E6020	Usually used with electrode negative or a.c. for fillets, and electrode positive or a.c. for flat welding
E6030	Usually used with electrode positive on d.c. or on a.c.

* Published in THE WELDING JOURNAL, September 1942.

It will be noted that for all grades, the tensile strengths and yield points are identical and that only the ductility requirements vary. These classifications are further determined by their use or application.

The structural steel welding codes of the AMERICAN WELDING SOCIETY specify that the electrodes for such work shall conform to the classifications shown in Fig. 5. These are commonly referred to as mild steel electrodes. Most of the welding is done with electrodes E6010, E6020 and E6030.

It is instructive to compare the tensile requirements for these electrodes with the base metal requirements in A.S.T.M. A7–42 Specifications for Steel for Bridges and Buildings.

	Plates, Sections and Bars
Tensile strength, psi	60,000 to 72,000
Yield point, min., psi	0.5 tensile strength
but in no case less than	33,000
Elongation in 8 in., min., %	$\frac{1{,}500{,}000^*}{\text{tensile strength}}$
Elongation in 2 in., min., %	22

* Other clauses of the Specifications reduce the percentage for plates under 5/16 in. and over 3/4 in. thickness.

Forms of Joints

Five common forms of joints are illustrated in Fig. 6. The names of each are descriptive and self-explanatory. In the next section are shown the types of welds that are applicable to these joints.

Types of Welds

The two fundamental types of welds are those shown in Figs. 7*A* and 7*B* and designated by the terms fillet and groove welds. The fillet type is used for all lap joints, and the groove type when the parts to be welded are in alignment. The examples of groove welds are a few of the more typical ones in common use. When designing butt joints it is necessary to give thought to the type of groove weld that is to be employed, in order that the total cost of joint preparation and of the welding shall be a minimum. Many of the electrode manufacturers furnish helpful information on this subject, but in the end the selection depends upon the facilities of the shop in which the work is to be fabricated. By way of illustration, the tabulation in Fig. 8 gives an approximate selection where welding is to be done from

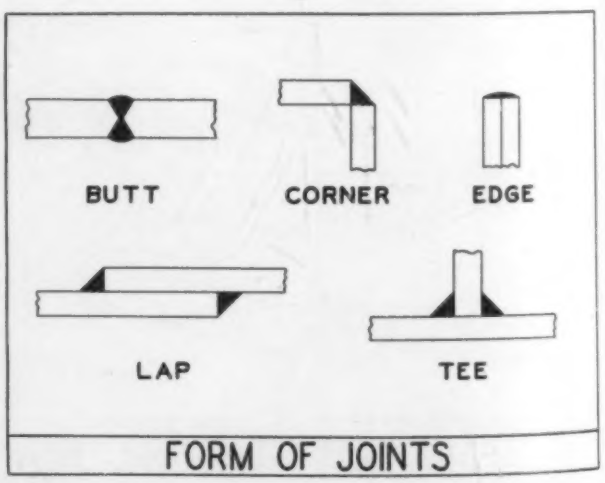

Fig. 6

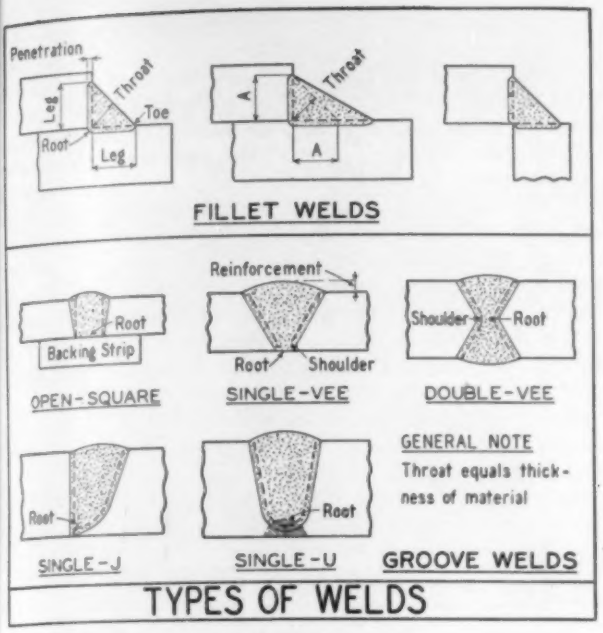

Fig. 7A

both sides. The J and U types were a development brought about by the excessive amounts of welding required with beveled joints in thick material.

It should be pointed out that such joints as the single-bevel, vee, single-J and single-U, are subject to distortion such as shown in Fig. 27 (*C*), due to the unequal shrinkage on opposite faces of the joint. When such a condition cannot be overcome by positioning the base metal parts to counteract the distortion, it may be desirable to use a symmetrical type of weld, such as the double-vee, double-bevel, double-J or double-U.

Certain terms relating to welds are so frequently found in welding literature that it may be well to review the definitions promulgated by the A.W.S. Committee on Definitions and Charts.

Base Metal (Parent Metal): The metal to be welded or cut.

Filler Metal: Material to be added in making a weld.

Weld Metal: The metal resulting from the fusion of the filler or base metals or both.

Bead Weld: A type of weld made by one passage of electrode or rod.

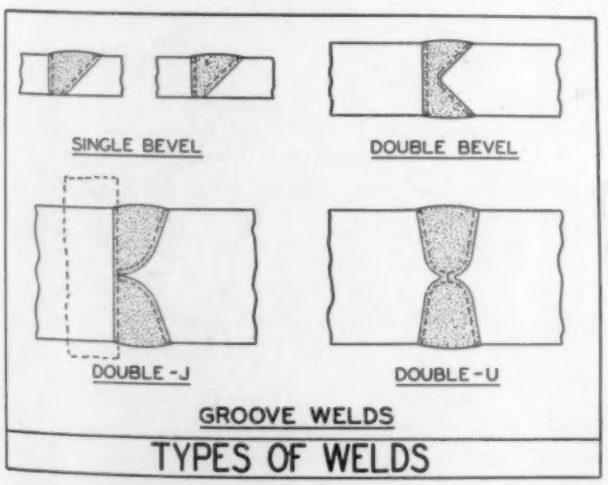

Fig. 7B

Penetration: The penetration, or depth of fusion, of a weld is the distance from the original surface of the base metal to that point at which fusion ceases.

Throat of Fillet Weld (Actual Throat): The distance from the root to the face of the weld.

Theoretical Throat of Fillet Weld: The distance from the root to the hypotenuse of the largest isosceles right triangle that can be inscribed within the weld cross section.

Reinforcement of Weld: Weld metal on the face of the weld, in excess of that required for the size of weld specified.

Size: (*a*) Fillet Weld—The size of a fillet weld is the leg length of the largest inscribed isosceles right triangle. (*b*) Groove Weld—The size of a groove weld is the depth of the groove. Where fusion materially exceeds the groove depth, the size of the weld is the depth of the groove plus the depth of fusion.

Effective Length: The length of the correctly proportioned cross section of a weld.

Thickness inches	Type of Joint	Sketch	Included Angle or Bevel
Up to 1/4	Open-Square		θ = 0° All Positions
5/16 - 1/2	Single-Bevel	θ	θ = 45° All Positions
1/2 - 1 1/4	Double-Bevel	θ	θ = 45° All Positions
1/2 - 1 1/4	Double-Vee	θ	θ = 60° All Positions
5/8 - 3	Single-J or U	θ θ	θ = 20° Flat and Overhead θ = 35° J / 45° U Vertical / Horizontal
2 -	Double-U	θ	θ = 20° Flat and Overhead θ = 45° Vertical Horizontal

GROOVED JOINTS

Fig. 8

These and a few other terms are illustrated in the sketches of Fig. 7*A*. The middle sketch for fillet welds shows the inscribed isosceles right triangle with the equal sides of length *A* when the legs of the weld are of unequal length. When the legs are of equal length, as in the left-hand sketch, the size is equal to the leg dimension.

Welding Positions

There are four common positions of welding that are shown and designated in Figs. 9 and 10. The flat position affords the greatest comfort to the welder and produces the best weld contour. Because the force of gravity aids in maintaining the weld metal in the desired position it is possible to attain higher speeds in welding than in other positions. In order to bring the welds into the flat position it is necessary to "position" the welds. There are machines on the market capable of handling

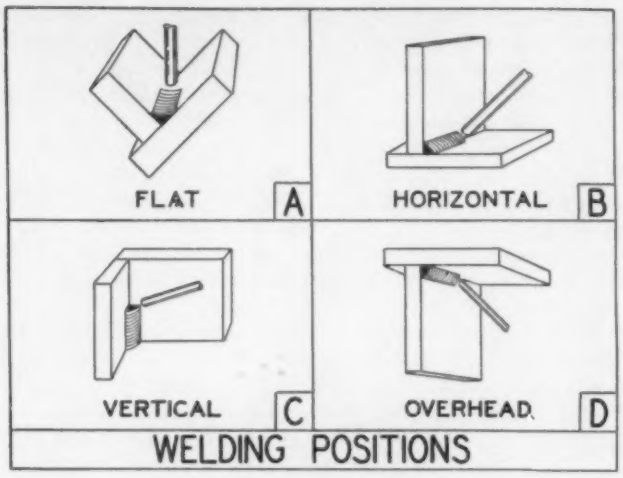

Fig. 9

weldments weighing several tons by means of which the weldments may be tilted or turned so that each line of welding is flat. It should also be noted that fillet welds made in the flat position have a curved face that will be shown later to present some desirable characteristics.

Next in ease of production for fillet welds is the horizontal position, distinguished from the flat position, only by the weld face being inclined to the horizontal plane. The vertical welding position presents increasing difficulty because greater care is required to keep the molten metal from running down and to maintain a triangular fillet without an excessive bulge or reinforcement.

The overhead position is the most difficult for fillet welds in which to secure penetration and to hold the molten metal in place against the force of gravity. It is also the most uncomfortable one for the welder and, for this reason, should be avoided whenever possible.

The welding positions for groove welds, shown in Fig. 10, are arranged in order of their difficulty of production—flat, vertical, overhead and horizontal.

Of course there are intermediate positions between those shown in Figs. 9 and 10. For example, in welding around a joint in a horizontal pipe, the position at the top is flat, on the sides, vertical and at the bottom, overhead. In order to distinguish when the position changes from horizontal to vertical and the like, a more elaborate method of classification is to be found in the A.W.S. Standard Qualification Procedure, published in THE WELDING JOURNAL of July 1941.

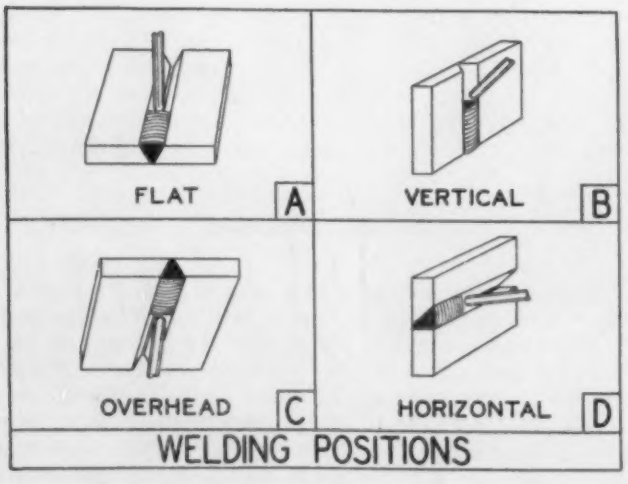

Fig. 10

Gas Cutting

Closely allied to welding is its counterpart, gas cutting —the severing of ferrous metals by means of the chemical behavior of oxygen in the presence of ferrous metals at high temperatures. The essential features of the process are illustrated in the sketches of Fig. 11. In the case of plain carbon steel, the metal is heated locally to a bright red color, when a jet of oxygen from an orifice in the cutting tip is introduced and the resulting controlled combustion removes the steel from a narrow kerf along the line of cut. The cutting tip is usually held vertical to the direction of travel, as indicated in the sketch of Fig. 11 (*A*), but across the line of travel the cutting tip may be held vertically for square cuts or tilted at an angle to produce cuts with beveled edges. This latter feature is particularly valuable in the preparation of butt joints for grooved welds.

Another application of gas cutting is shown in Fig. 11 (*B*) where the tip is tilted at such an angle to the line of

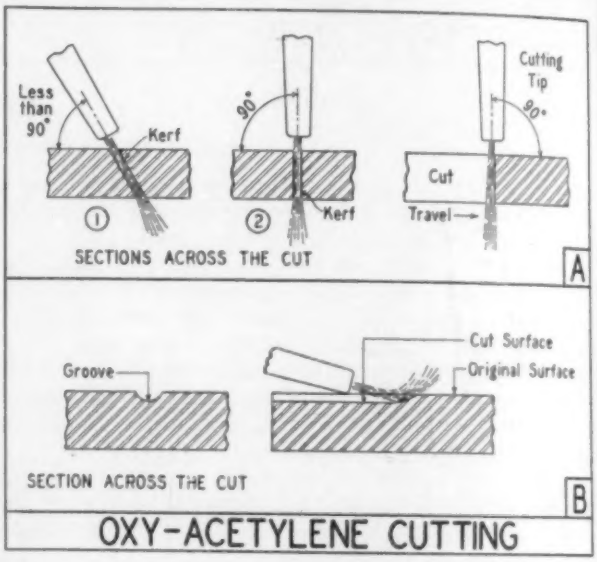

Fig. 11

travel that the cutting action does not extend through the material, but removes the metal in a manner to produce a groove on the surface. The shape and size of this groove can be controlled by altering the angle of attack and by varying the oxygen pressure. This type of cutting is a flame machining operation called "flame gouging" and is opening up new fields of application for gas cutting.

The photographs of Fig. 12 illustrate the character of cuts made under three common conditions of guiding the tip. With free-hand flame cutting of material 2 in. and under, the cut surface may be expected to come within a tolerance of $\frac{1}{8}$ in. either side of the line of cut. When the tip is held against the edge of a template, or mounted on wheels or rollers (free-hand guided) the cutting can be kept within a tolerance of $\frac{1}{16}$ in. either side of the line of cut. For more exacting requirements, there are machines which hold the cutting torch in a firm position and guide its movements by accurate templates. Under such controls, it is possible to produce cuts with the following tolerances:

Thickness of Material, In.	Tolerance of Cut, In.
1	0.003
2	0.010
4	0.0175
6	0.0312

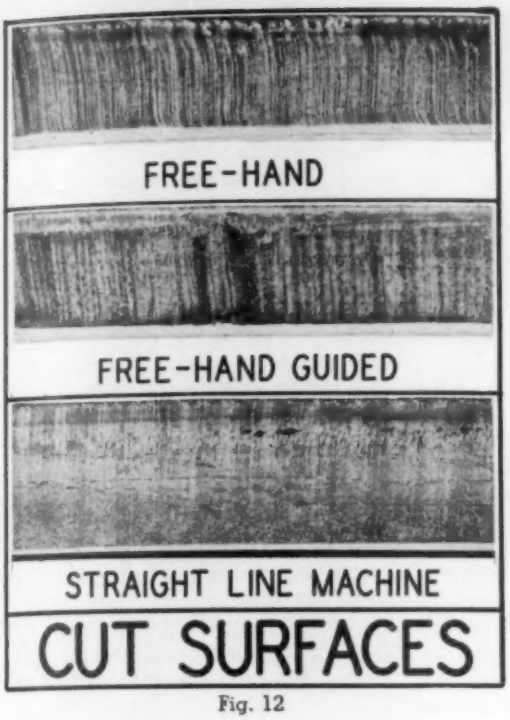

Fig. 12

In the early days of welding, there were questions as to the desirability of welding on flame-cut edges without any further treatment of the edges after cutting. This was due to some doubts as to the effect of hardness produced by the rapid heating and cooling of the metal, but tests and practical experience have long since dispelled these doubts in the case of ordinary structural steel. The Building Code and Bridge Specifications of the AMERICAN WELDING SOCIETY permit the practice of welding on flame-cut surfaces, preferably prepared with a mechanically guided torch, although well-executed manually guided cuts are accepted after inspection.

When higher carbon steels (above 0.35 C) or alloy steels are flame cut, it becomes necessary to exercise special precautions, such as preheating and annealing of the cut edges, and such work should be done only after consulting recognized authorities or published literature on the subject.

Qualification Tests

Qualification tests comprise one of the most important factors in assuring welds of high quality and adequate strength. From the first, it was recognized that every welder should be required to demonstrate his fitness and ability before being permitted to weld on the finished product. Likewise, it was realized that the welding procedure should be tested, for without a satisfactory procedure even a skilled welder may have difficulty.

In the early days of welding there was a multiplicity of qualification tests which worked a hardship on contractors, but the need for uniformity in requirements that would be widely accepted finally brought about a Standard Qualification Procedure, prepared by Committees of the AMERICAN WELDING SOCIETY and published in THE WELDING JOURNAL of July 1941. It is divided into two parts: Part I, Procedure Qualification; Part II, Operator Qualification.

An investigation of a welding procedure is concerned with such items as the chemical composition and mechanical properties of the base metal, and filler metal, sizes of electrodes, the current and arc voltage, the type of groove, the position for welding and in some cases the preheating and postheating treatment. Under the requirements of Part I, the tests are intended to determine the tensile strength, ductility and degree of soundness of welded joints under a given Procedure Specification. Standard specimens and their manner of testing are specified for both groove and fillet welds and the required results are set forth. Once a procedure has been qualified, requalification is required only when certain stated factors have been changed.

The Operator Qualification of Part II is based on the premise that simple tests may be used to determine an individual's ability to make a sound weld when a satisfactory procedure has been established. The tests used are as follows:

(*a*) For Groove Welds:
- (1) Root-Bend Test
- (2) Face-Bend Test
- (3) Side-Bend Test

(*b*) For Fillet Welds:
- (1) Fillet-Weld-Soundness Test

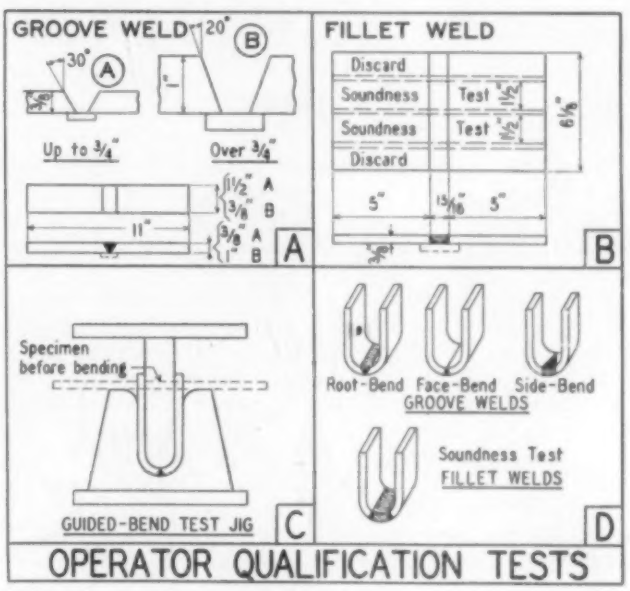

Fig. 13

The specimens for groove welds are shown in Fig. 13 (*A*). For material up to 3/4 in. thickness the groove of Type A is used in a plate of 3/8 in. thickness from which 1 1/2 in. wide specimens are cut. When the welder will be working on material over 3/4 in. thick he is required to make specimens in 1-in. plate, using groove B. In that case the specimens removed are 3/8 in. wide so that they can be bent in the same jig as the specimens cut from the 3/8 in. plate.

The details of the specimens for the fillet-weld-soundness test are shown in Fig. 13 (*B*). Two fillet welds are laid down and then the remainder of the joint is filled as indicated by the cross hatching.

The welder is required to weld the test specimens in each of the four positions (flat, horizontal, vertical and overhead) in which he will subsequently be working.

The specimens are to be bent in a jig substantially in accordance with the sketch shown in Fig. 13 (*C*). The specimen is first laid flat on the lower die as indicated by the dotted lines. Then the upper die, or plunger, forces the specimen downward, bending it into a U-shape. When removed from the jig the specimens have the forms depicted in Fig. 13 (*D*). The groove weld tests

are named according to the portion of the weld that is on the outside of the bend, as is clearly evident from the sketches. For the fillet weld specimen the portion that was next to the backing up strip (which is removed before testing) is on the outside of the bend.

Any specimen in which a crack or other open defect is present after the bending, exceeding $^1/_8$ in. measured in any direction, is considered to have failed. Quoting the Standard Qualification Procedure: "A specimen shall be considered as having passed if it has cracked or fractured and the fractured surface shows complete penetration through the entire thickness of the weld, and absence of slag inclusions and porosity to the extent that there are no gas pockets or slag inclusions exceeding $^1/_{16}$ in. in greatest dimension and the sum of the greatest dimension of all such defects in any square inch of weld metal area does not exceed $^3/_8$ in. (If necessary, the specimen shall be broken apart to permit examination of the fracture.)"

It will be noted that the Operator Qualification aims only at determining the ability of the individual to produce sound welds. Matters relating to the strength of welds are determined under the Procedure Qualification. The logic of this separation of requirements is entirely reasonable.

Inspection

No simple method of testing welds, comparable with "tapping rivets" has been developed. There are several methods which work best with groove welds and are, therefore, not quite as applicable to the fillet welds commonly used in structural work.

The most effective and also the most costly methods are the X-ray and the gamma-ray by which photographs of the welds are taken, utilizing the penetrating radiations of the X-ray tube and of radium, respectively. There can be no doubt of the efficiency of either method in locating the character and extent of any defects. For Class I unfired pressure vessels the A.S.M.E. Boiler Code makes the X-ray examination of welds one of the requirements for their acceptance after fabrication.

Another non-destructive test is that known as Magnaflux inspection. The method depends upon the behavior of magnetic lines of force as they flow through the piece of ferromagnetic material under inspection. The magnetic flux is created by disposing electric cables or suitable apparatus in the proper relation to the part to be inspected and passing either a.-c. or d.-c. current through the system. When the flow of the lines of force is interrupted by a surface crack in the material, some of the magnetic flux passes out of the material and travels across the gap in the air, resulting in a field of local flux leakage. If subsurface discontinuities are not too far beneath the surface and are of sufficient magnitude, they will divert the flux and some of it may still be forced into the air. The general behavior of the lines of force is quite analogous to that of internal stress flow around discontinuities, a subject that will be discussed in a later section of this paper under "Stress Paths."

The inspection medium consists of a finely divided ferromagnetic material applied either dry, as a powder, or in a liquid suspension. The dry method is used almost exclusively throughout the welding field. Direct current of suitable amperage is applied, generally by means of electrode type contacts, to a limited length of weld and the dry powder applied over the magnetized area while this current is passing. Wherever there is a leakage field, as over a surface crack or a subsurface defect of sufficient extent, the powder is attracted and held, thus forming a pattern of the crack. The results of the test depend upon the interpretation of the pattern and the appearance is of fundamental importance. The powder is generally of a very light pearl gray color so that the pattern is clearly visible on dark steel surfaces. On bright surfaces it is possible to use black or red powders.

In spite of the fact that the application of Magnaflux to weld inspection is not as simple a matter as its application to the location of surface defects and that it may not reveal some defects in welds, it is still a valuable, inexpensive and comparatively simple non-destructive test. For a detailed description of the process and its varied applications in industry, the reader is referred to the book, *Principles of Magnaflux Inspection*, by F. B. Doane.

For structural welding, visual examination by a competent inspector during and/or upon completion of the welding is satisfactory, but in addition there are several very definite things which can and should be done to insure workmanship of good quality.

First, none but properly qualified welders should be permitted to do structural welding, or any welding where strength is a requisite consideration. The investigation of the Structural Steel Welding Committee showed that, in general, the higher a welder's qualification rating, the better was his subsequent performance on the program test specimens.

Second, a definite procedure control should be followed. This involves the preparation of the parts to be welded, the selection of the proper electrodes or welding rods, the technique of the welding process and provision for adequate supervision. Reputable manufacturers of welding equipment and supplies are, naturally, vitally interested in the successful application of their products and many of them maintain large service organizations to which the user may turn for information and guidance. It is well to remember that proper design and adequate instructions on drawings covering welding procedures and sequence are also important factors in promoting good workmanship and quality.

These two steps will go a long way toward the production of sound welds of the required strength. There is left the inspection of the finished weld. Observation of a welder at work will furnish information as to the general character of his welds. If arc welding, he should hold a short arc and be able to maintain it without frequent interruptions. It will also be possible to judge the amount of penetration which he is obtaining. One should note whether the joint surfaces are clean and free from dirt, paint or grease, which interfere with the welding process. The inspector need not watch the welder continuously; an occasional observation is sufficient.

Visual inspection will yield information to the trained eye of the engineer or inspector. All welds should be gaged for size. When butt welds are reinforced, it is customary to specify that the reinforcement shall not exceed $^1/_8$ in. in height. Several forms of fillet weld gages are shown in Fig. 14. Since the design strength of welds is based upon the throat dimension it is desirable to check the welds for throat size. In sketches *A*, *B* and *C* we have two forms of gages shown as applied to welds with theoretically correct throat sizes, while in sketch *D* is illustrated a weld with reinforcement. The second dimension of a fillet weld that needs to be checked is the size or length of leg. Sizes called for on drawings and in designs refer to the minimum or theoretical dimensions and in practice these sizes may overrun by $^1/_{16}$ in. Hence most gages are provided with minimum and maximum limits as shown in Sketches *E* to *H*.

A number of essential points to be observed in the visual examination of welds are shown in the sketches of Fig. 15, illustrating various weld contours. Sketches *A*

to *E* show the application of gages to fillet welds, with appropriate comments below each sketch. It is evident that the first three weld sections have throat dimensions meeting the requirements, while the last two fail to meet the requirements.

The surface of the weld should be smooth with regular and even ripples, caused by the steady advance of the electrodes or welding rods. The edges of the weld should merge with the base metal thoroughly. Sketch *F* indicates a form of weld which may occur when the operator fails to secure full penetration, with a resulting overhang. Often the point of a knife or the edge of a sharp chisel can be inserted under the weld. Drops of metal quite generally spattered on the base metal indicate that too long an arc was probably held with the consequent likelihood of porosity and inferior penetration.

Sketch *G* depicts a common and undesirable form of weld that occurs when too much heat has been used in order to get speed in welding. Undercutting can be a serious defect due to the reduction in cross-sectional area of the member, and furthermore it acts to produce concentration of stress—a subject which will be treated more fully in a later section of this paper dealing with "Stress Paths." The sketches for groove welds indicate points to be observed and the comments on fillet welds are equally relevant.

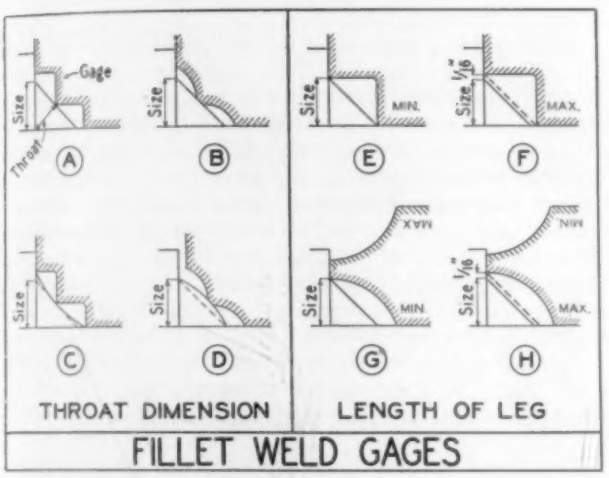

Fig. 14

No amount of printed instructions can take the place of actual experience with practical welding operations. One has to see welding done, both in the shop and in the field, in order to have a working understanding of the process and its application.

Ultimate Weld Failures

It is valuable information to know how welds fail when they are stressed to their ultimate carrying capacity. A few examples of typical cases will suffice to illustrate the usual manner of failure.

In Fig. 16 are shown several types of welded test specimens which can be imagined as having been given tension tests to produce failure. Sketch *A* is representative of welds subjected to longitudinal shear. Failure will usually occur through the critical throat section unless there is an extensive lack of fusion on one of the legs of the fillet, in which case failure may take place between the base metal and the weld. End welds under transverse shear also fail through the throat as indicated in sketch *B*. There is a condition with end welds, as shown in sketch *C*, in which the eccentricity of the bars causes a moment on the weld that forces failure to occur along the vertical leg of the fillet.

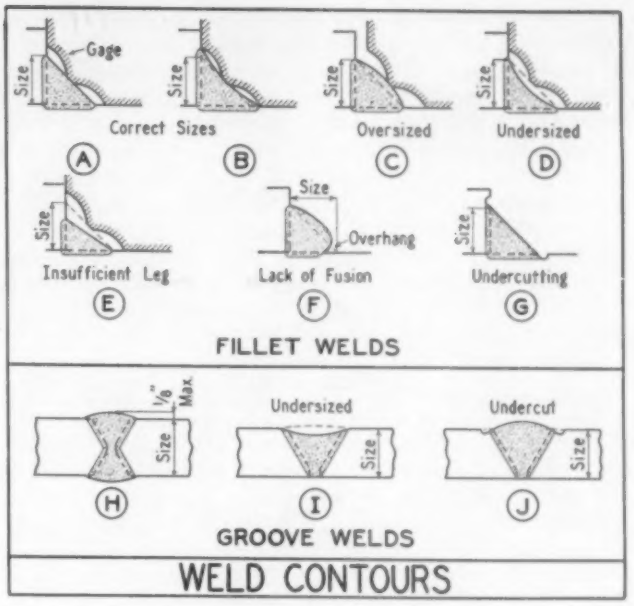

Fig. 15

Groove-welded specimens behave in two ways, depending upon the reinforcement. With covered electrodes it is usual to produce welds of greater strength than the base metal of ordinary structural steel. If the reinforcement is of sufficient amount and is left on the specimen, failure will probably occur in the base metal close to the weld, due to the stress concentration or notch effect at the junction of the edge of the weld and the base metal. Should the reinforcement be removed by milling or grinding, then failure is likely to occur in the base metal at some distance from the weld.

Eccentricity and its accompanying prying effect on welds is an important factor in the strength of fillet welds. Figure 17 illustrates the effects of flexure on fillet-welded joints. In sketch *A* it should be noted that the weld is subjected to a prying effect which is resisted by internal forces having a small lever arm. When

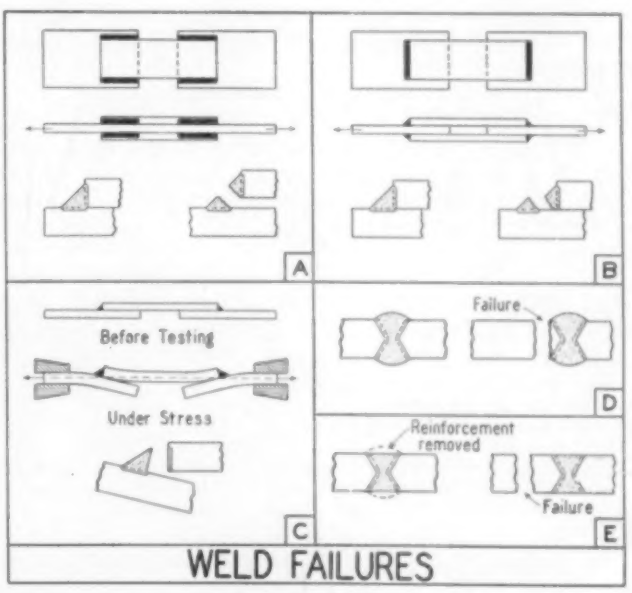

Fig. 16

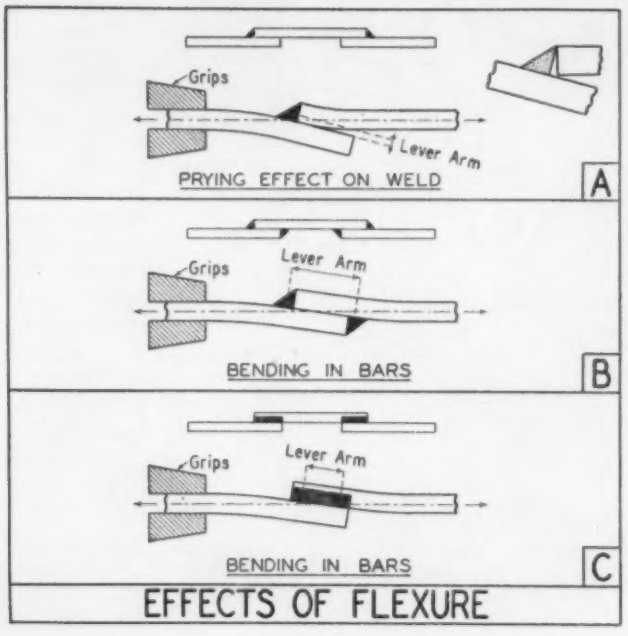

Fig. 17

welds are placed on the ends of both bars, as in sketch *B*, the lever arm is materially increased and the bending is thrown more largely into the bars. The same is true of joints with side welds, typified by sketch *C*.

As the throat is the critical section of both fillet and groove welds, it has become the practice to base the safe working unit stresses upon this section, as will be explained presently.

American Welding Society Building Code

Every structural engineer should be acquainted with the Code for Arc and Gas Welding in Building Construction, formulated by the Committee on Building Codes of the AMERICAN WELDING SOCIETY. The Code proper deals with materials, permissible unit stresses, design, workmanship, inspection and qualification, while several appendices contain explanatory information and data.

For the purposes of this paper, it will be useful to quote the following articles from Part II (Permissible Unit Stresses) of Section 2 (Design of Welded Connections) of the Code:

"*204. Unit Stresses in Welds*

(*a*) Welded joints, constructed with the base metal and filler metal prescribed in this Code, shall be proportioned so that the stresses caused therein by the loads specified in the Building Code shall not exceed the following values:

Kind of Stress	*Permissible Unit Stress Pounds per Square Inch*
Shear on section through throat of fillet weld, or on faying surface of filled plug or slot weld	13,600
Tension on section through throat of butt weld	16,000
Compression (crushing) or shear on section through throat of butt weld	Same as corresponding allowable stress for base metal

(*b*) Fiber stresses due to bending shall not exceed the values prescribed above for tension and compression, respectively.

(*c*) Stress on a fillet weld shall be considered as shear on the throat, for any direction of the applied stress.

(*d*) Neither filled nor fillet-welded plug or slot welds shall be ascribed any value in resistance to stresses other than shear.

205. Combined Stresses

In the case of fillet welds subjected to both shear and bending stresses, the maximum unit stress resulting from the combination shall not exceed that allowed for shear in Art. 204; in the case of butt welds it shall not exceed the tension or compression value (whichever is the smaller) allowed in Art. 204.

213. Eccentricity

In designing welded joints, adequate provision shall be made for bending stresses due to eccentricity, if any, in the disposition and section of base metal parts and in the location and types of welded joints."

American Welding Society Bridge Specifications

A companion document to the Building Code is the "Specifications for Welded Highway and Railway Bridges" prepared by the Conference Committee on Welded Bridges of the AMERICAN WELDING SOCIETY. Every structural engineer should be thoroughly familiar with these specifications, for they treat the subject of fatigue and repeated stresses in a most comprehensive manner and present a set of working unit stresses based upon the ranges of maximum and minimum stresses. There are several helpful appendices, particularly the one dealing with unit stresses for welded design, which is replete with illustrative examples of design calculations.

Again it will be useful to quote the following articles from Part II (Permissible Unit Stresses) Section 2 (Design of New Bridges) of the Specifications:

"*209. Maximum and Minimum Stresses*

Maximum and minimum stresses (axial stress, bending moment, shear, etc., respectively) shall be computed in accordance with the requirements of the applicable general specifications. These will hereinafter be referred to as "Max." and "Min.," respectively. "Max." refers to the numerically greater stress, of whichever sign, and *is to be used* in the design formulas of this Section *with a plus sign*. "Min." refers to the numerically smaller stress; *if it be of the same sign as "Max." it shall be used with a plus sign, and if it be of opposite sign to "Max." it shall be used with a minus sign*, in the designing formulas.

211. Required Weld Areas

The required effective weld areas shall be calculated in accordance with Formulas 7 to 9 of Table 2. But in the case of highway bridges, for all members in which the maximum stress is derived from a critical loading of more than two panels, or 60 linear feet of bridge, whichever is greater, "Min." shall be disregarded in Table 2 and the formulas containing "Max." alone shall be applied.

217. Effective Areas of Weld Metal

(*a*) The effective area of butt and fillet welds shall be considered as the effective length times the effective throat thickness.

(*d*) The effective length of a fillet weld shall be the over-all length, including returns, less twice the weld size.

Table 2—Weld Areas

Formula No.	Type of Weld	Type of Stress	Required Effective Weld Area, A (see Note 1)
7	Butt	Tension or Compression	$A = \frac{\text{Max.} - 1/2\ \text{Min.}}{13{,}500}$ but not less than $\frac{\text{Max.}}{16{,}000}$
See Art. 212 (*c*) for penalty in case of single-vee backed-up welds.			
8	Butt	Shear	$A = \frac{\text{Max.} - 1/2\ \text{Min.}}{9000}$ but not less than $\frac{\text{Max.}}{12{,}000}$
See Art. 212 (*c*) for penalty in case of single-vee backed-up welds.			
9	Fillet	Tension, Compression or Shear	$A = \frac{\text{Max.} - 1/2\ \text{Min.}}{7200}$ but not less than $\frac{\text{Max.}}{9600}$

NOTE 1: See Article 212 (*b*) for allowable stresses in butt welds made smooth and flush.

(*e*) The effective length of a butt weld shall be the width of the part joined, assuming the ends of the weld are made as specified in Art. 607 (*e*). A transverse skewed butt weld shall not be assumed in computations to be longer than the width of the joint or piece, perpendicular to the direction of stress.

(*f*) The effective throat thickness of an equal leg 45° fillet weld shall be considered as 0.707 times the nominal size of the weld.

(*g*) The effective throat thickness of a butt weld shall be the thickness of the thinner part joined, with no allowance for reinforcement."

Report of the Structural Steel Welding Committee

While the Report of the Structural Steel Welding Committee of the American Bureau of Welding is long since out of date (having been presented in 1931), the *Engineering News-Record* said of it that "in its conception and execution this investigation has seldom been equaled in the field of industrial research." It played a prominent and influential part in the development of structural welding and deserves to be known to all engineers who desire a breadth of knowledge of welding. This report can still be obtained from the AMERICAN WELDING SOCIETY.

Stress Distributions in Side Fillet Welds

The stress distributions in side fillet welds received a very considerable amount of theoretical and experimental investigation at the hands of early investigators. The late Professor J. Hammond Smith of the University of Pittsburgh seems to have been the first to carry out a series of careful laboratory tests. These were followed by a mathematical analysis by H. W. Troelsch, published in the *A.S.C.E. Proceedings*, November 1932.

It had been surmised for some time that the stress was not evenly distributed throughout the length of a side fillet weld. Professor Smith showed that its ends are more highly stressed than the central portion and Mr. Troelsch's analysis demonstrated the same fact.

Figures 18 (*A*) and 18 (*B*) show the theoretical distribution along the welds of two test specimens in the program of the Structural Steel Welding Committee. In the charts, the vertical ordinates represent the intensity of stress on the basis of the average stress being equal to unity. Referring to Fig. 18 (*A*), it will be noted that the ends have a stress from five to six times that of the central portion. (The variation in values at the ends is due to unequal areas in the component bars—causing higher concentration at the smaller bar.) The maximum stress is 2.44 times the average. It will also be seen that there is a large portion of the weld which has less than the average. Figure 18 (*B*) is a similar chart for a specimen with only 3-in. welds and again we have the same general distribution but with less variation.

Any wide difference of distribution, such as here shown, attracts immediate attention and might possibly lead one to hesitate to use such welds. It should be pointed out that all the results, theoretical and experimental, apply only within the elastic limit of the material. What undoubtedly happens is that yielding takes place within the steel bars and to a somewhat less extent within the weld itself. The result must be a flattening of the curve of stress distribution.

An interesting paper by A. Hrennikoff, Esq., of the Dominion Bridge Company on a similar investigation of the distribution of stress among rivets in a single line was published in the November 1932 *Proceedings* of the A.S.C.E. Figures 18 (*C*) and 18 (*D*) are charts of the theoretical distribution. The results are strikingly parallel to those for side welds. It has long been recognized that the end rivets were highly stressed so long as the material was not stressed beyond the elastic limit or slip of the rivets. Here again, the yielding action must produce a more uniform division of the stress among the individual rivets. Certainly no structural engineer hesitates to put several rivets in a line and calculate their number on the basis of equal strength.

The investigations do have a practical significance when one is designing a joint to carry impact or repeated stresses. Such loads might induce an initial fatigue failure at the extreme ends of the weld, followed by progressive failure with repetition of the loading conditions. This subject will be discussed at greater length in a later section on Fatigue.

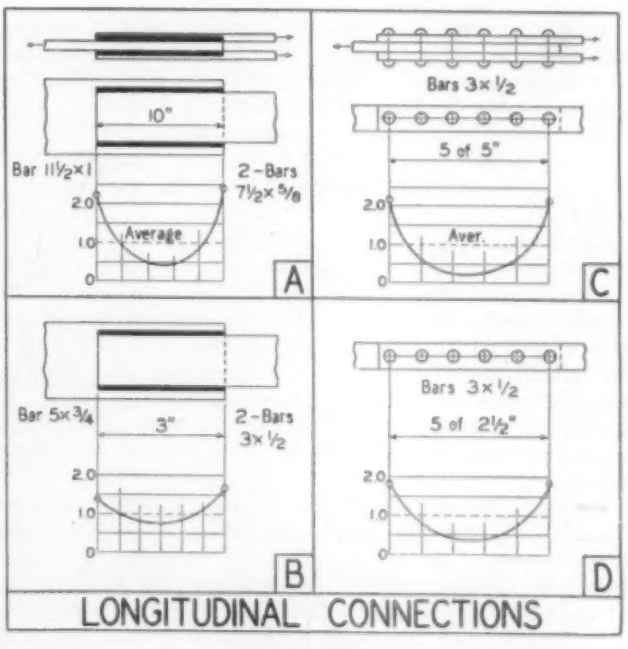

Fig. 18

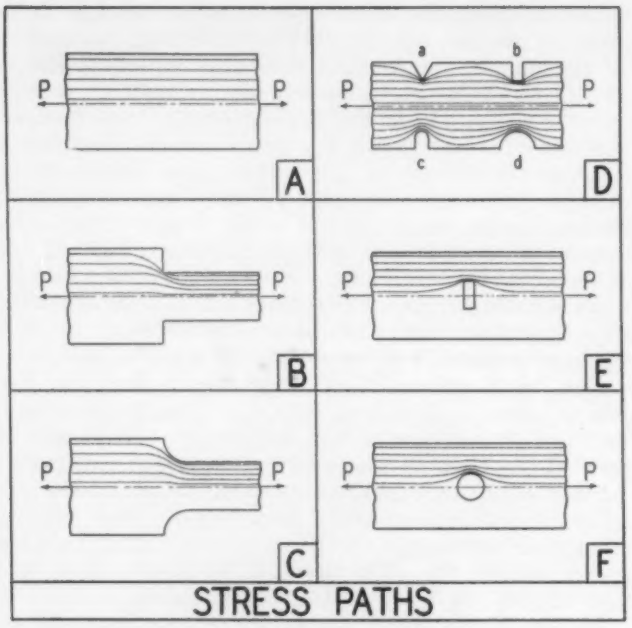

Fig. 19

Stress Paths

The inherent rigidity of a welded joint, brought about by the actual fusion between the weld and base metals, has made the subject of stress distribution a matter of vital importance in the design of welded construction. The designing engineer must have a thorough knowledge of the manner in which stresses are transferred or travel through welded joints.

Photoelasticity has contributed greatly to the solution of many problems in stress distribution and has become a potent instrument of research. It is an optical method of stress analysis, employing polarized light to produce visible stress patterns in models of transparent celluloid or bakelite to which various types of loading may be applied. An excellent book entitled *Photoelasticity* by Dr. Max M. Frocht of the Carnegie Institute of Technology appeared in 1941. It is clearly written and profusely illustrated and will provide any interested reader with a good understanding of the subject.

It is not possible to treat this topic in detail within the limits of this paper, but some of the essential features will be briefly discussed. Let us begin with sketch *A* of Fig. 19 which may be taken to represent a portion of a bar loaded uniformly across its width by tensile forces *P*. We may think of the internal stresses traveling along the uniformly spaced lines indicated in the upper half of the sketch.

In sketch *B*, the cross section of the bar is suddenly altered in width and consequently the even flow of stress is interrupted and the paths become crowded at the sharp corner, as shown. Photoelastic investigations have developed the fact that the intensity of stress at the corner may be several times the average uniform intensity at some distance from the corner. This discontinuity of section is known as a "stress raiser," since it causes an increase of intensity or a stress concentration.

The stress concentration factor can be reduced in the manner shown in sketch *C* by introducing a rounded fillet in place of the sharp corner. This permits of more even flow of stress with less intensity of stress. The larger the radius, the better will be the conditions.

The essence of the subject of stress concentration is contained in Figs. 19 (*A*) to 19 (*C*). Let us now apply the information to a few conditions encountered in welded joints. First, we shall examine some discontinuities that may occur in practice. In sketch *D* of Fig. 19 are shown various notches which can interrupt the smooth, uniform flow of stresses within the material. Notches *a* and *b* have sharp, angular corners which cause high stress concentrations in the manner indicated in sketch *B*. Cracks, even minute ones, are like notch *b*, except that the two sides of the notch are practically in contact. Scratches from grinding or machining, nicks from a chisel or marks from steel lettering stamps create notches which are frequently the initial causes of failure. The notches *c* and *d* lack the angular corners and approach the conditions of sketch *C*, yet are productive of stress raisers.

Sketch *E* shows a rectangular hole (such as produced by lack of root welding in double-vee welds) in the path of the stresses and the results so far as stress distribution is concerned are similar to the conditions prevailing in sketch *B*. The round hole of sketch *F* produces conditions like those of sketch *C*.

Through the courtesy of Dr. Frocht, the photographs of Figs. 20 and 21 are reproduced from his book on photoelasticity. The fringe patterns around the notches with rounded fillets are clearly shown in Fig. 20 and illustrate the stress concentrations. Below the photograph is a plot of the stress intensity measured across the section *XX*, with unity representing the average intensity for uniform distribution. For the particular specimen tested, the maximum stress is 3.0 times the average. The stress concentration factor is said to be equal to 3.0.

In Fig. 21 we have the fringe pattern around a circular hole in a bar, corresponding to Fig. 19 (*F*). Even with the relatively smooth edges of the hole, there is a resulting stress concentration factor of 2.3. The regions of high stress are of limited width.

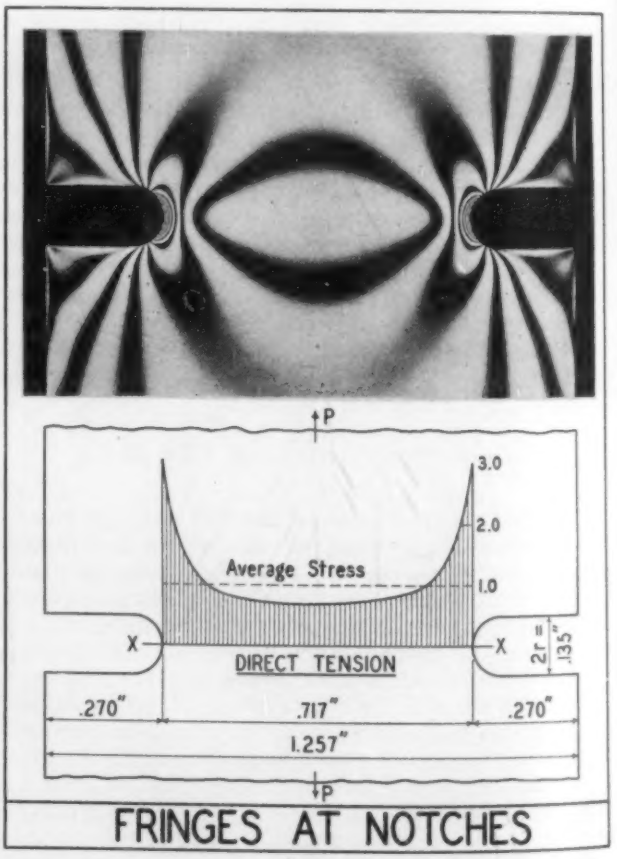

Fig. 20

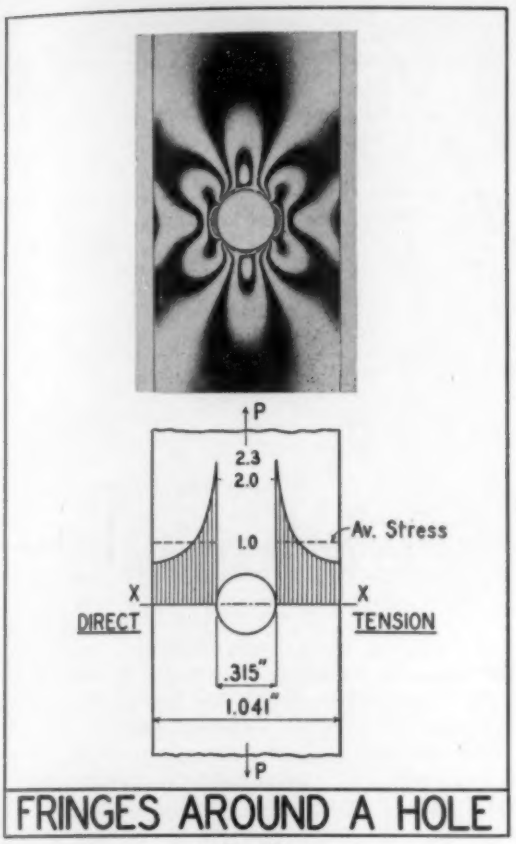

Fig. 21

In contrast to groove welds which generally lie in the direct path of the stress, fillet welds always lie at a point of discontinuity or offset in the path of stress transfer. The stress paths in end fillet welds are shown in Fig. 22. Sketch *A* represents a weld with a heavily reinforced contour. The paths in the weld take the shortest route with resulting concentrations at points *a* and *b*. It will be observed that the contour of the weld face makes an angular notch at point *a*, and thereby creates an aggravated condition similar to that in Fig. 19 (*B*). Attention should also be called to the fact that the reinforcement serves no useful purpose and is, rather, a detriment.

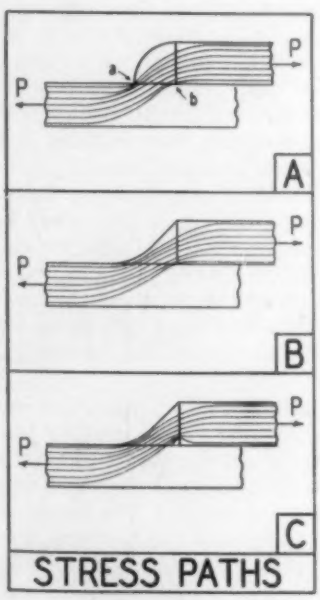

Fig. 22

By forming the weld contour with a curved face at the toe of the weld, as in Fig. 22 (*B*), a smooth path is created, the notch effect is alleviated and the stress concentration is greatly reduced. Such weld contours may be produced by welding in the flat position of Fig. 9 (*A*), with results similar to the contour in Fig. 15 (*B*). It is also possible to improve conditions by making the horizontal leg of the fillet longer and thus give a flatter slope to the weld face.

Lack of fusion and penetration at the root of a fillet weld can cause a serious stress concentration as illustrated in Fig. 22 (*C*). Thorough penetration at the root of such welds is, therefore, of prime importance.

Various theories or methods of calculation have been proposed for obtaining the numerical value of the stress developed in end fillet welds, that is, welds which are transverse to the direction of the stress. None have been really satisfactory. After all, the main reliance of the engineer must be tests to determine the actual loads that welds can carry. Many results of such tests are available and, with the proper factor of safety, the designer can rely upon the working unit stresses to give satisfactory structures. The complicated stress rela-

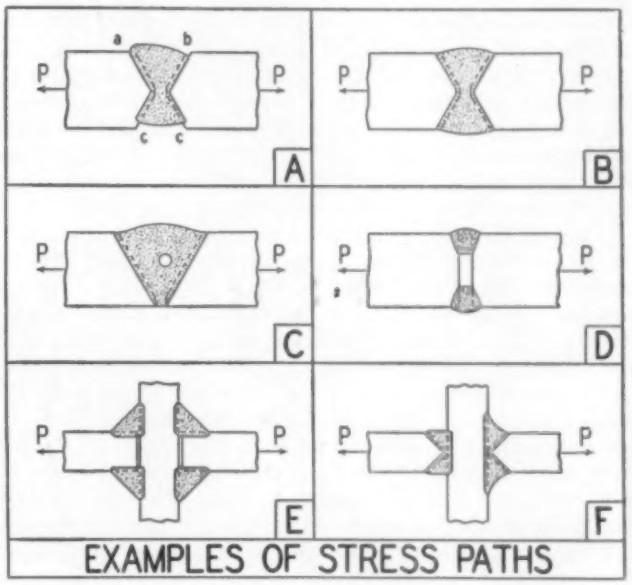

Fig. 23

tions which undoubtedly exist within the small compass of a weld make any attempts at exact analysis difficult, if not impractical.

Figure 23 delineates some examples of stress paths and stress-raisers. The groove weld of sketch *A* might be termed a horrible example of what to avoid in such a weld. The notch effect at point *a* caused by the excessive reinforcement, the undercutting at point *b* and the angular notches at points *c* (together with the lack of complete filling of the groove on the under side) are all creators of stress concentrations. By contrast, the groove weld of sketch *B* with its moderate reinforcement and smooth contours at the edges provides an ideal path for stress travel.

Sketch *C* represents the presence of a gas pocket or a slag inclusion within a weld which becomes a stress raiser in the same manner as the hole in Fig. 19 (*F*). The condition of Fig. 19 (*E*) can readily arise in the welded joint of Fig. 23 (*D*) and concentration of stress is inevitable at the angular corners where the fusion ceases. It is obvious that this condition would be eliminated by complete fusion across the joint.

Figure 23 (*E*) illustrates a tee joint and it is clear that here again there is a rectangular opening similar to the hole in Fig. 19 (*E*), with the inevitable stress concentrations. Sketch *F* shows two joints with the fusion complete, thus eliminating the internal stress raisers, but a distinction between the two joints illustrated should be noted. Under the action of the tensile forces *P*, the transfer of stress in the left-hand joint meets an angular discontinuity at the junction between the weld and the vertical member, making stress concentration a certainty. The right-hand joint, in effect, has a fillet weld in each corner which provides a smooth path for the stresses to spread out into the vertical member with a consequent minimizing of the "stress-raiser" effect.

The A.W.S. Specifications for Welded Highway and Railway Bridges prohibits the use of one-sided connections. Examples of such connections are shown in Fig. 24. From the preceding discussion it is easy to trace the stress paths and to recognize the serious notch effects and stress raisers.

This subject of stress paths and stress concentrations has been discussed at some length because of its practical importance. Fortunately, for all concerned, the situation with structural steel is not so alarming as it sounds, although flagrant violations of the principles set forth have led to serious trouble. The ductility of steel of structural grade steps in to modify the effect of these stress concentrations. When the steel is first stretched, there may be some points of stress concentration where the unit stress reaches the yield point. When this occurs, the high stresses will be prevented by the yielding of the steel, causing a more uniform distribution to take place and thus modifying the seriousness of the situation.

To have knowledge about the behavior of welds is to be forearmed in dealing with them. It should be remembered that the working unit stresses which engineers use are based upon many tests of actual welded joints. We have seen, for example, that fillet welds have a stress path that is productive of stress concentrations, but this effect has been present in the tests, and the final strength of the test joints has been influenced accordingly. The fillet weld is a very useful one and very widely employed and we need only to recognize its particular peculiarities and make the appropriate allowances for them. Knowing that good root penetration and the avoidance of sharp-cornered toes on such welds are important factors, the endeavor should be made to achieve these desirable results. Each type of weld has its individual characteristics and requires individual consideration.

A sense of balanced judgment needs to be exercised in dealing with defects. It is not so much the existence of blowholes, for example, that is important, as their size,

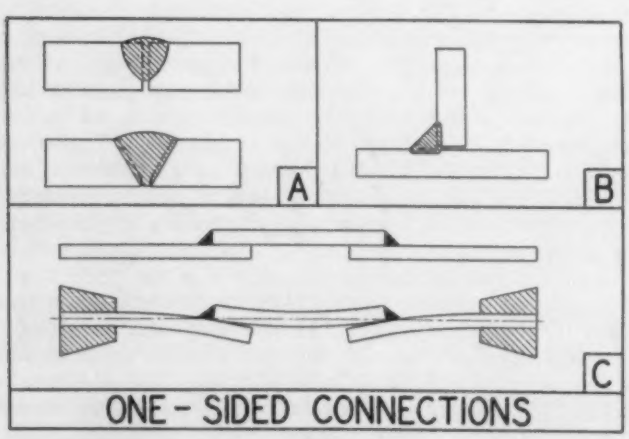

Fig. 24

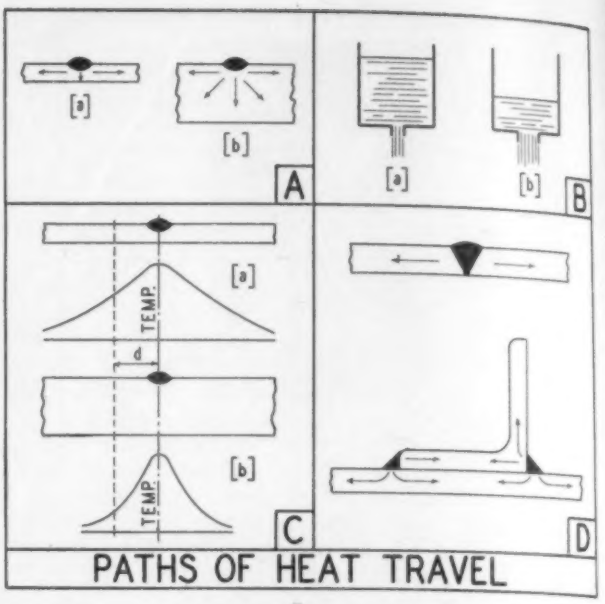

Fig. 25

location and the number that exist within a certain length or cross section of the weld. In such structures as pressure vessels, it is so important to have knowledge concerning interior defects that the specifications for several types of these vessels require every weld to be examined with X-rays.

The service to which a structure is to be subjected has a bearing upon this subject. In those that carry only steady loads, slowly applied and removed, as in an office building, warehouse or hospital, the matter of stress concentration is not so vital as would be the case where impact and repeated stresses are present as in bridges, cranes, derricks, machinery and material-handling equipment. All such factors must be properly taken into account by an engineer when designing any structure.

Temperature Effects

The behavior of steel with changes of temperature is of vital importance in welding operations. While many of the problems arising in welding from the heat effects of the operation are in the realm of fabrication, the engineer can minimize some of the difficulties to the extent that he understands the actions due to changes of temperature and makes the proper adjustments in his design.

Expansion of steel with increasing temperature is inevitable and if resisted results in internal stress. Contraction with decreasing temperature is likewise to be expected and when resisted results in internal tensile stress. The coefficient of expansion for structural steel is 0.0000067 per 1° F. A temperature change of 150° F. results in a change of length in the steel of 0.001 in. per inch—equivalent to the change produced by a unit stress equal to the yield point of the steel. This is an important fact to bear in mind when considering the effects produced by the cooling of welds and adjacent base metal.

The transfer of heat in steel is another important factor as it affects the cooling rate—another factor which will be considered in a subsequent paragraph. In Fig. 25 (*A*) are shown a thin plate in sketch *a* and a thick plate in sketch *b*. It is clear that the heat has broader and more extensive avenues of escape from the welding bead on the thicker plate, just as water in vessel *b* of Fig. 25 (*B*) will escape more rapidly when the same volume is poured into each. If we measured the temperature in

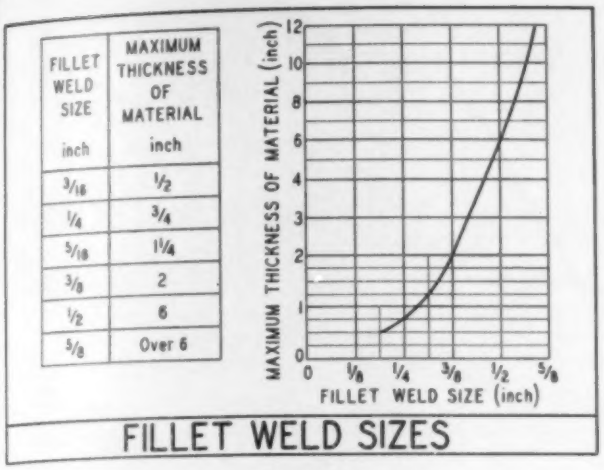

Fig. 26

the plates of Fig. 25 (*A*) at any given instant after welding we would find temperature gradients similar to those in Fig. 25 (*C*), indicating a more rapid cooling or chilling in thicker material.

A comparison between butt welds and fillet welds is made in Fig. 25 (*D*). In the former, there are only two avenues of escape for the heat of welding. Since still air is a poor conductor of heat most of the welding heat spreads outward in the steel and of course ultimately escapes into the air by radiation. In the case of fillet welds on toes of angles or edges of overlapping plates, it will be seen that there are three paths of heat travel and at the heel of an angle there may be four. Hence fillet welds are more subject to rapid cooling than are butt welds and in heavy material both types of welds may cool more rapidly.

The rate of cooling has an important bearing upon welding, more particularly for alloy and high carbon steels than for the ordinary steel of structural grade. When the latter steel is heated to about 1500° F. or above, it is composed of grains of a single constituent having a crystalline structure to which the name "austenite" is given. As the steel cools slowly from this temperature the austenite transforms to a dual constitution, iron and iron carbide, called pearlite, which is the crystalline structure in structural steel developed in cooling from the rolling temperature. The rate of cooling may be increased to a definite point and still have all of the austenite changed to pearlite. The increased rate of cooling will be accompanied by an increase in the hardness of the steel and a finer layered structure in the pearlite.

However, when the rate of cooling is greatly increased, there is not sufficient time for the transformation to pearlite to be completed and another hard crystalline structure, known as martensite, may begin to form at temperatures below 500° F. With still more rapid cooling, we may have no pearlite at all but mostly martensite, which is seriously lacking in ductility and is an undesirable constituent of the metal in a welded joint. With increase in the carbon content of the steel, a slower rate of cooling is required if martensite is not to be formed. Fortunately, for structural welding, the regular grade of steel has such a low carbon content that the rate of cooling is not so critical, in most instances, as to be serious. It cannot be neglected, however. Based upon practical experience, both the Building Code and the Bridge Specifications of the AMERICAN WELDING SOCIETY contain a standard of fillet weld sizes in relation to the thickness of material to be welded. The table

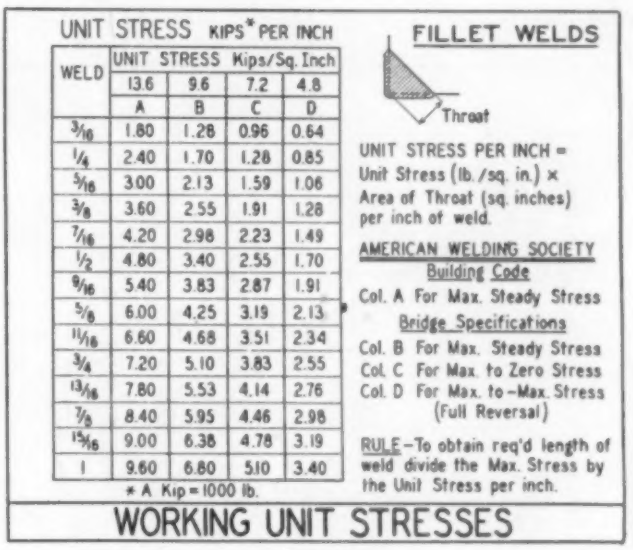
UNIT STRESS KIPS* PER INCH
WELD
UNIT STRESS Kips/Sq. Inch
13.6 9.6 7.2 4.8
A B C D
3/16 1.80 1.28 0.96 0.64
1/4 2.40 1.70 1.28 0.85
5/16 3.00 2.13 1.59 1.06
3/8 3.60 2.55 1.91 1.28
7/16 4.20 2.98 2.23 1.49
1/2 4.80 3.40 2.55 1.70
9/16 5.40 3.83 2.87 1.91
5/8 6.00 4.25 3.19 2.13
11/16 6.60 4.68 3.51 2.34
3/4 7.20 5.10 3.83 2.55
13/16 7.80 5.53 4.14 2.76
7/8 8.40 5.95 4.46 2.98
15/16 9.00 6.38 4.78 3.19
1 9.60 6.80 5.10 3.40
* A Kip = 1000 lb.
FILLET WELDS
Throat
UNIT STRESS PER INCH = Unit Stress (lb./sq. in.) × Area of Throat (sq. inches) per inch of weld.
AMERICAN WELDING SOCIETY
Building Code
Col. A For Max. Steady Stress
Bridge Specifications
Col. B For Max. Steady Stress
Col. C For Max. to Zero Stress
Col. D For Max. to -Max. Stress (Full Reversal)
RULE - To obtain req'd length of weld divide the Max. Stress by the Unit Stress per inch.
WORKING UNIT STRESSES

Fig. 28

and chart for their requirements are given in Fig. 26. These sizes insure a weld of sufficient strength to avoid cracking during shrinkage.

Shrinkage in welded construction is certain and makes itself evident in the distortions with which everyone is all too familiar. Figure 27 illustrates the distortions that occur in a few simple joints when the base metal parts are free to move. From what we have observed about the unit stresses set up by contraction and expansion, it is clear that we shall stress the weld metal beyond the yield point if we endeavor to straighten the joints, either by force or by placing welds on the opposite side to balance the construction.

As one studies the problem of shrinkage stresses, there seems little doubt but that they are high, often exceeding the yield point. Practical experience has definitely shown, however, that in most cases, when welding on steel of structural grade, these stresses are not serious, for the reason that brittle internal structures are not formed in steel of structural grade if the cooling rate is not too sudden. With pearlite structures, the steel is soft and ductile, the yield point is well below the ultimate strength and, if a high contraction force threatens, the metal is able to stretch and dissipate high stresses.

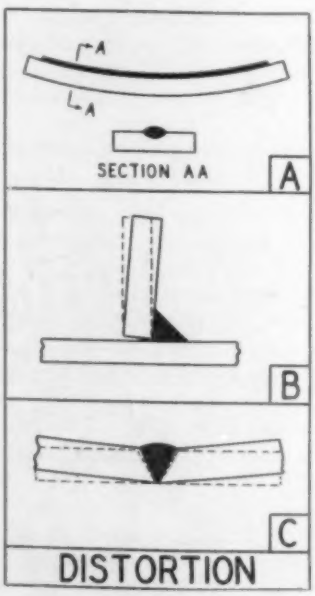

Fig. 27

There are times when we wish to have stress-free joints in our structures before they go into service. Structures subject to dynamic shock or fatigue, pressure vessels and some kinds of machinery are examples. One method of achieving this result is by peening, but a much more certain method of stress relief is that of heating the welded construction in an annealing furnace to a temperature of 1100 to 1200° F., and subsequently cooling it slowly, either in the furnace or still air.

For a more extended discussion of the subject of "Temperature Effects," the reader is referred to an article by the writer entitled "Engineering Essentials for Welders," published in THE WELDING JOURNAL of April 1942, from which this discussion has been condensed.

Unit Stresses for Fillet Welds

In designing connections for fillet welds it has been found much more convenient to use the unit stress per inch rather than the unit stress per square inch as stated in the A.W.S. Building Code and Bridge Specifications—portions of which were quoted earlier in this paper. Accordingly, the table in Fig. 28 has been prepared,

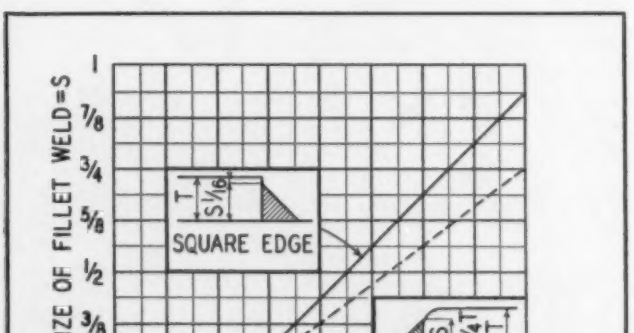

Fig. 29

giving the unit stress (kips per inch) based upon some of the principal unit stresses per square inch specified in these two documents.

The Building Code deals only with steady stresses for which we have the unit stresses in Column A of the table. Three conditions of stress are tabulated for the Bridge Specifications. Under Column B, are the values to be used when the maximum stress is steady. When a member may have a low stress or no stress under dead load and receives a maximum stress with the application of the live load, we have the unit stresses in Column C. These unit stresses are based on Formula No. 9 in Table 2 in which the minimum stress equals zero, a condition of variation in stress from none to a maximum. Column D covers the condition of full reversal of stress from a maximum tension to an equal compression. It is interesting to observe that the Bridge Specifications give a lower value to fillet welds under steady load than that permitted under the Building Code. The effect of varying stress is reflected in the lower unit stresses per-

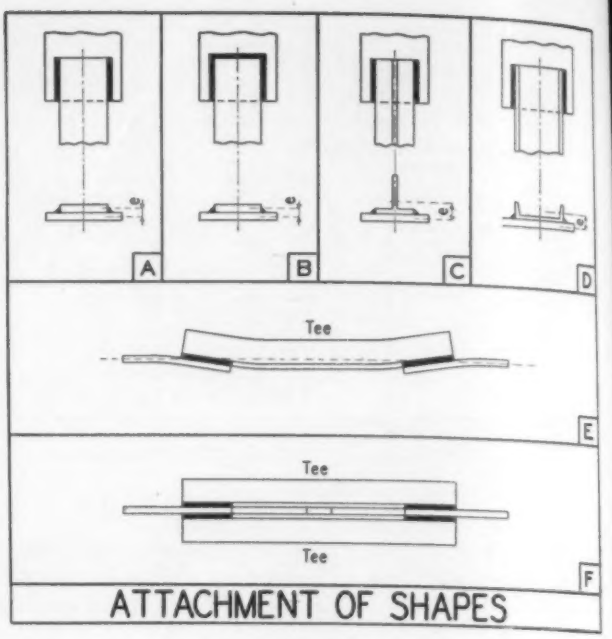

Fig. 30

mitted as the conditions approach full reversal. The table will be found helpful in the design of all fillet-welded connections.

A practical consideration in connection with the size of fillet welds is set forth in the chart of Fig. 29. When a fillet weld is placed on the square edge of a plate or the round toe of a structural shape or angle, there is always difficulty in obtaining a weld having a size equal to the thickness of the material, unless special effort is made to build up the weld. In the case of the square edge, the upper corner of the plate melts and flows downward into the general contour of the weld. For the rounded toe it is impossible to secure a throat equal to that of a weld the same size as the thickness unless the toe is built up by the addition of welding. Consequently, it is specified that the maximum weld sizes shall be those shown in the sketches on the chart, unless special attempts are made to build up the welds.

Attachment of Structural Shapes

The attachment of members which are symmetrical about an axis parallel to the line of stress presents no serious problem. Figure 30 shows a few typical examples. The length of the welds should be equal on

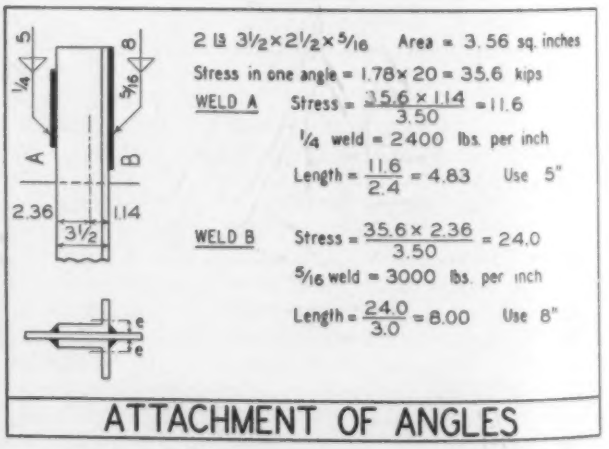
5
1/4
8
3/16
A
B
2.36
1.14
3 1/2
e
e
2 ⊔ 3 1/2 × 2 1/2 × 5/16 Area = 3.56 sq. inches
Stress in one angle = 1.78 × 20 = 35.6 kips
WELD A Stress = 35.6 × 1.14 / 3.50 = 11.6
1/4 weld = 2400 lbs. per inch
Length = 11.6 / 2.4 = 4.83 Use 5"
WELD B Stress = 35.6 × 2.36 / 3.50 = 24.0
5/16 weld = 3000 lbs. per inch
Length = 24.0 / 3.0 = 8.00 Use 8"
ATTACHMENT OF ANGLES

Fig. 31

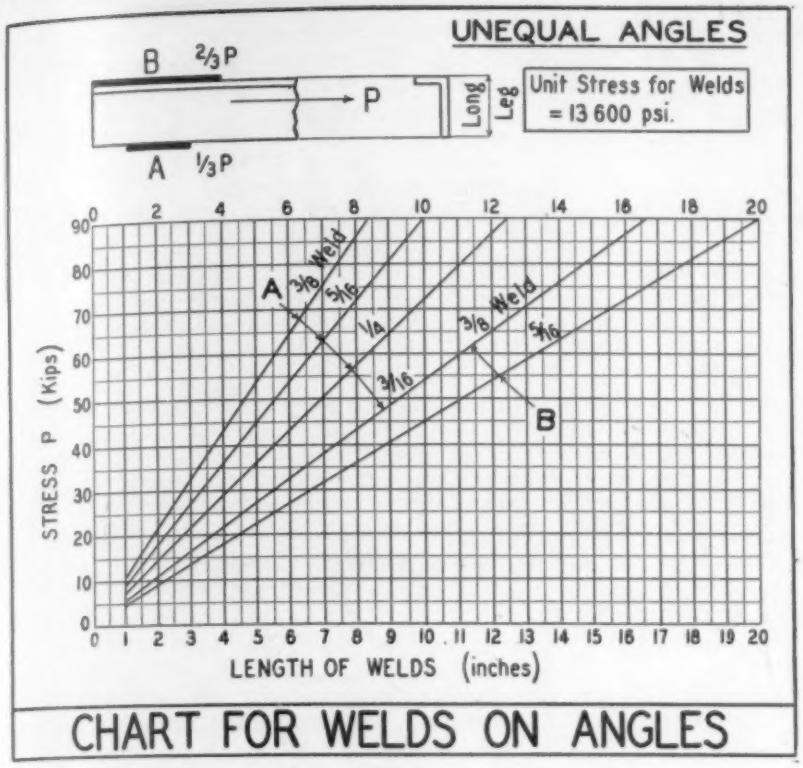

Fig. 32

both edges and is determined by dividing one-half the total stress in the member by the unit working stress per inch of weld. On some occasions, for lack of space, it is necessary to place welding on the ends of a member as shown in sketch *B*. The unit stress per inch for end welds is the same as for side welds. In such cases the value of the end weld strength is deducted from the total force, the remainder being divided equally between the side welds.

When the member is composed of a single structural shape, as in sketches *A* to *D*, there is a certain eccentricity, *e*, the effect of which is to produce the condition shown in sketch *E*. The member tends to distort until its center of gravity lies on the line of pull. For this reason, it is the general practice to reduce the unit stress in the member itself.

In a double member, sketch *F*, each part resists the tendency of the other to bend so that we do not have the high bending stresses of the unsymmetrical member.

Attachment of Angles

The generally accepted method of calculating the length of welds required to attach angles is given in Fig. 31. Assuming 2Ls $3^1/_2 \times 2^1/_2 \times {}^5/_{16}$, the total allowable stress in one angle equals the area times the unit stress, i.e., equals $1.78 \times 20{,}000 = 35{,}600$ lb. The stress is divided between welds *A* and *B* inversely proportional to their distances from the center of gravity of the angles. The calculations are illustrated in the figure. The size of weld *A* is governed by the rolled edge of the angle and was determined from the chart of Fig. 29. The ${}^5/_{16}$-in. size for weld *B* is that generally obtained with ${}^3/_{16}$-in. covered electrodes.

There is an eccentricity on the welds due to the center of gravity of the angles being a distance, *e*, from the back of the angles. It is desirable to keep this distance as small as possible and it has been the writer's practice to use unequal legs with the shorter one outstanding. With double-angle tension members it is essential to have separators to prevent their inward deflection, and in struts to reduce the L/r ratio of the individual angles.

It has long been recognized in structural design that single angles in tension should not be used to their full cross-sectional value. A clause in the A.R.E.A. General Specifications for Steel Railway Bridges reads as follows:

"The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50 per cent of the area of the unconnected leg."

In welded construction there are no holes to be deducted for the net area of the connected leg. An approximation to the A.R.E.A. provision for the common sizes of unequal legged angles would be to take 80% of the gross area as effective.

Other factors in use vary from 60 to 75% of the gross area. For angles with unequal legs the area in the longer leg equals approximately 60% of the total area of the angle, so that one rule might be set up by which only the attached leg of a single angle member in tension shall be considered effective. It has been the general practice to calculate the welds for single angles in the same way as for double angle members as outlined above.

The center of gravity of angles with unequal legs is located about one-third the length of the longer leg from the heel of the angle. This fact makes feasible the construction of the chart of Fig. 32 in which the total stress *P* in the angle is divided in the proportions of one-third to weld *A* and two-thirds to weld *B*. Basing the length of welds upon the unit stress of 13,600 psi. the chart gives those lengths for varying sizes of welds and total stress. The sizes of weld *A* may be obtained from Fig. 29. We may check the calculations in Fig. 31 and the resulting lengths will be found to be in agreement.

Eccentric Connections

The accepted practice for designing eccentric connections is given in Fig. 33. Assume a weld of length, *L*, to be carrying a load, *W*, acting at a distance, *a*, from the line of weld. The resultant stress on the weld is a combination of the vertical and horizontal components. The problem is simplified by treating only the linear dimension of the weld and then using a stress per linear inch instead of per square inch. The vertical component equals the load divided by the length of the weld, while the horizontal component equals the moment, Wa, divided by the section modulus of the weld, $L^2/6$. In the formula for the resultant stress, *R*, on the weld, it will be observed that the quantity within the brackets is constant for a given problem and has been given the value *K*.

In order to facilitate the calculation of eccentric connections, the chart of Fig. 33 has been prepared, the use of which will be illustrated by the following example: Assume $W = 45$ kips and $a = 4$ in. With a ${}^5/_{16}$-in. weld, having a working unit stress, *R*, of 3.0 kips per inch (see table in Fig. 28 under Column A), the value of *K*

is found to be 3.0/45 = 0.067. Entering the upper half of the chart at K = 0.067 and following vertically down to the curved line for $a = 4$, we find that L should equal 22 in. A beam bracket with two such welds could sustain a load of 90 kips.

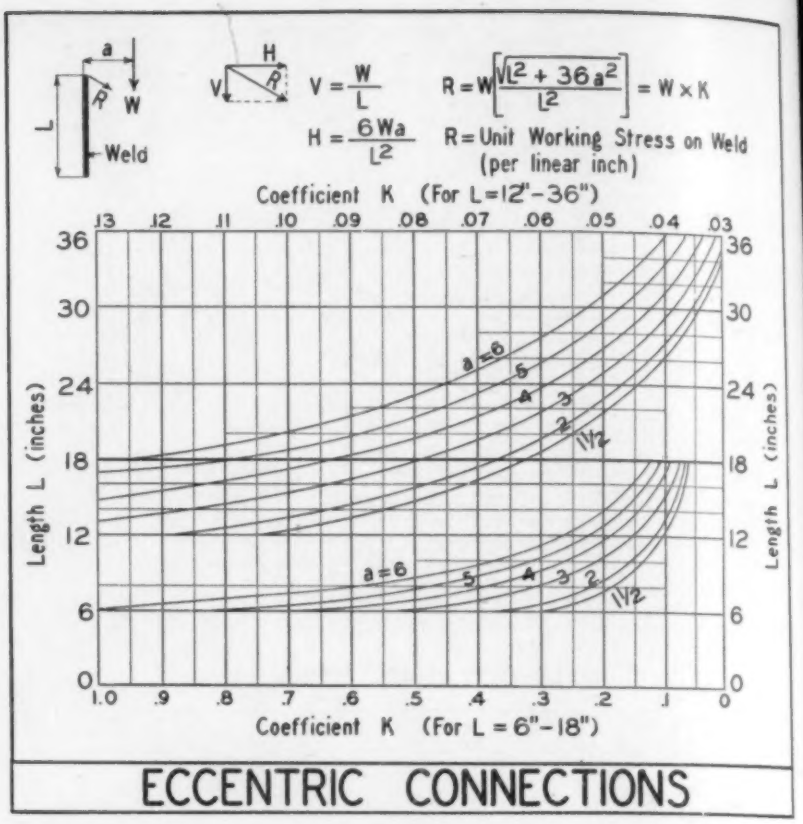
a
W
L
Weld
H
V
R
V = W/L
H = 6Wa/L²
R = W√[(L² + 36a²)/L²] = W × K
R = Unit Working Stress on Weld (per linear inch)
Coefficient K (For L=12"-36")
.13 .12 .11 .10 .09 .08 .07 .06 .05 .04 .03
36 30 24 18 12 6 0
a=6 5 4 3 2 1½
Length L (inches)
1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1 0
Coefficient K (For L = 6"-18")
ECCENTRIC CONNECTIONS

Fig. 33

General Design Considerations

The chief difference in design between welded and riveted construction lies in the details of the connections. Welded joints are more rigid and require special study to avoid harmful effects which can follow from this condition. This discussion, therefore, will be devoted largely to the joints of structures.

Early attempts at designing welded members consisted mainly in the straight substitution of welds for rivets. This was not satisfactory, practically or economically. A fresh approach to the subject has developed better methods. As an example of this development, it might be cited that tees with deeper stems than those rolled as tee-sections were found to be very desirable and from the early practice of splitting I-beams along the center of the web to obtain such tees has come the furnishing of these split beams direct from the rolling mills.

It still remains that "a cardinal aim should be simplicity in the details." Study spent on the details before a job goes to the shop can effect worth-while savings in cost of welded construction. One fundamental difference from riveted construction must be constantly kept in mind. The shop assembly of riveted members is guided and facilitated by the punched holes.

When welding is used, the aim is to avoid holes and the work is held together by clamps or jigs until the welds can be made. Where there is duplication in reasonable quantity, it is often advantageous to build jigs which serve both to facilitate the assembly of the several component parts of a structure without measurements or layouts and to hold those parts in proper relation to each other until the welding is completed. Jigs are often as important a study as the structure itself.

Much of the layout work is done directly on the steel without the usual templates. Dimensions should be given in such a way as to readily assist this part of the work.

Symmetry in the location of welds and welded connections is very desirable. The heat of welding and shrinkage upon cooling is apt to cause a certain amount of warping, as illustrated in Fig. 27, and symmetry will many times assist in maintaining the parts in their original or intended position or shape. The effect of a weld on one side of a member is to cause the member to take a curved shape, concave toward the weld. This condition is often avoided in the shop by setting the parts before welding in a position contrary to the expected warping, so that the final position of the parts is the one desired, or by securing the parts against bending during fabrication.

The relative ease of welding in various positions has been discussed in an earlier section of this paper. The designing engineer should bear that discussion in mind for it affects not only the quality and uniformity of welds but also the cost. Overhead welding is difficult, both from the point of view of welding technique and from the uncomfortable position of the welder and furthermore, in the field, will usually require some form of staging or platform for the welder to stand upon. It is very desirable to plan the design and the details of construction so that field connections may be welded in a down position, if at all feasible. Alterations and additions to existing structures may often require overhead welding, in which case it should be kept to a minimum.

The erection of structural steel requires some means of holding the members together until the field welds are placed. For members framing to columns, it is general practice to utilize a positive type such as a bolt or drift pin. Minor connections have been made with clamps or wedges and in some instances without any temporary means. Examples of some of the methods used will be illustrated in a later part of this paper.

One principle of economical fabrication is to avoid, so far as possible, both welding and punching operations on the same piece of main material. It is expensive to handle heavy pieces through several shop operations and an economy can be effected by punching the necessary holes in the detail material which is welded to the large-sized members.

Detail Design Considerations

The common sizes of electrodes for hand welding are $^5/_{32}$ and $^3/_{16}$ in. diameter, with $^1/_4$ and even $^5/_{16}$ in. diameter permitted when welding in the flat position. A $^5/_{16}$-in. fillet weld is most common in structural work for the reason that it is about the maximum size of weld that can be made in one pass with a $^3/_{16}$-in. covered electrode. Large sizes of fillet welds have to be made in two or more passes of the electrode in arc welding. In the gas-welding process any size can be made in one pass.

The selection of the type of joint depends upon the

service of the structure, as discussed under the subject of "Stress Paths" and upon the relative total cost of the joint. The AMERICAN WELDING SOCIETY has a Committee on Standard Procedures of Welding and Standards for Welded Joints which has prepared a Tentative Recommended Procedure Specification for Metallic Arc Welding. When this report is approved and released for publication by the SOCIETY it will make available a wealth of data on the design of joints, the sizes of electrodes to be used, the number of passes and the recommended currents and voltages to be used for the electrodes in such classifications as those shown in Fig. 5.

One point in connection with the use of larger sizes of fillet welds needs to be emphasized. The strength of welds is approximately in proportion to the throat dimension and consequently to the size, while the cross-sectional area of the weld varies as the square of the size. Figure 34 (*A*) illustrates certain facts in regard to size in which a $^5/_{16}$-in. weld is taken as a basis for comparison.

The area of the cross section of a weld is a measure of the amount of weld metal to be deposited. To go from a $^5/_{16}$-in. to a $^1/_2$-in. weld increases the strength by the factor 1.60 while the weld metal required 2.56 times as much. Weld metal is deposited at a uniform rate, hence the total time of welding increases more rapidly with larger welds. The effect on the cost is evident. The cost of depositing a pound of weld metal varies very little for the usual sizes of welds. Therefore, it is obvious that the ratio of unit stress to cross-sectional area should be kept large.

The minimum size of fillet welds for strength purposes should be limited to $^3/_{16}$ in. and the length of any weld should not be less than four times its size. For example, the minimum length of a $^5/_{16}$-in. weld should be $4 \times {}^5/_{16} = 1^1/_4$ in. With butt welds the least permissible length is commonly taken as four times the throat dimension. Where a butt weld lies at an angle to the line of stress its length shall not be assumed in computations to be longer than the width of the piece or joint, perpendicular to the direction of stress.

The distribution of welding is often of prime importance when it relates to the ability of a joint to withstand the handling to which it is subjected in shipping and erection. A connection which is entirely adequate to perform its final function in the structure may be wholly unsatisfactory for handling.

Figure 34 (*B*) will clearly bring out the difference between poor and good construction. When a member is welded on one side only, the connection has very little resistance to a force acting in the direction of the arrows. By placing welds on the opposite edge, an almost surprising amount of stiffness is gained. A case came to the writer's attention in which an angle on top of a beam lintel, welded continuously along the heel of the angle for weather tightness, was satisfactorily stiffened for shipment by placing welds 2 in. long about 5 ft. 0 in. centers on the toe of the angle.

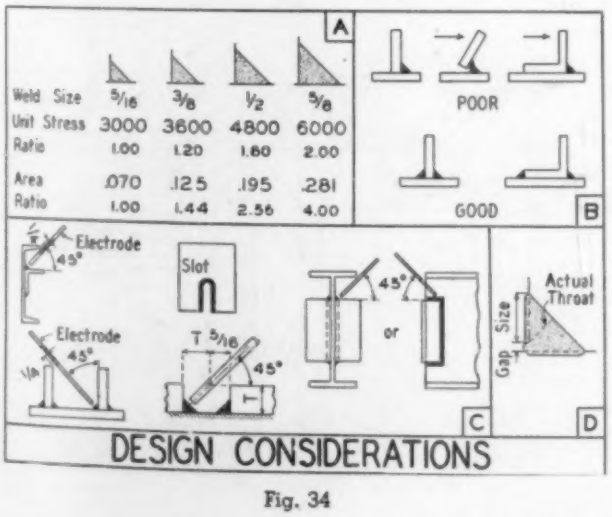

Fig. 34

No design will be completely satisfactory unless attention is given to arranging the details so as to provide sufficient clearances for performing the welding. Just as there are considerations in riveted work which affect gages and spacing of rivets, so there are certain points which should govern in welded construction. First of all the welder must be able to see his work clearly. Secondly, it must be possible to hold the electrode or welding rod in the proper position to secure penetration and sound quality in the welds. Failure to observe these two fundamental rules may slow production and lead to difficulty in achieving satisfactory welds. A few hours of personal observation of welders at work is much more valuable experience for the designer than simply reading about these points.

A few typical cases, shown in Fig. 34 (*C*), will illustrate the requirements for arc welding. The requirements for gas welding are of a similar character, bearing in mind that space must also be provided for manipulating the welding torch.

The ideal position of the electrode when welding material of approximately the same thickness is at 45° to the legs of the fillet. It is a simple matter to make a quarter scale layout of joints when necessary to determine the clearance. A practical rule for the width of slots is to make the opening equal to the thickness of the material plus $^5/_{16}$ in.

One practical remark should be made about welding across a small gap. When such a case arises, the size of the weld must be increased by the amount of the gap in order to provide the designed size of the throat. Figure 34 (*D*) shows this condition. The A.W.S. Bridge Specifications (Section 603 (*a*)) provide that if the separation is $^1/_{16}$ in. or greater, the leg of the fillet weld shall be increased by the amount of the separation, but that in no case shall the separation be permitted to exceed $^3/_{16}$ in. It should always be the endeavor to assemble the parts with as close contact as practicable.

Other important considerations have been brought out in other sections of this paper. These primary objectives for good welding design will be reinforced by the designer's own experience in this field.

Symbols

Those who were connected with the design of welded structures in the early stages of the development felt the immediate need of some method of designating the welds on the drawings. Welding, expanding rapidly into many fields, needed a much more elaborate system of notation than that shown in the earlier paper by the writer. The system in general use today throughout the industries of this country is given in Fig. 35, taken from the 1942 edition of "Welding Symbols and Instructions for Their Use," published by the AMERICAN WELDING SOCIETY.

A word should be said about some of the illustrations in this paper, to make clear that the symbols are sufficient in themselves to indicate the position of the welds, but for the sake of added clarity, the welds have also been shown. By so doing, it is hoped that the reader may more readily interpret the text and gain a familiarity with the correct symbolization. For a complete knowledge of the application of the symbols, the reader should have reference to the A.W.S. publication, which is replete with illustrative examples.

Butt-Strap Splices

An excellent paper by Professors Hollister and Gelman of Purdue University entitled "Distribution of Stresses in Welded Butt Strap Joints," was presented at the 1932 Fall Convention of the AMERICAN WELDING SOCIETY. Figure 36 gives the general results of their investigation on five types of splices with double straps. The increment of load for which the stresses were determined by means of Huggenberger extensometers was 50,000 lb. On the basis of a uniform distribution, the stress in the plates is as follows:

Main plate (all types) = 16,670 psi.
Butt straps (Types A, C, D and E) = 14,290 psi.
Butt strap (Type B) = 16,670 psi.

It is readily apparent that where some of the welding is transverse to the line of stress, as in Types C, D and E, there is a more uniform distribution of stress on sections *AA* and *BB* with decreased intensity. Type E gave the best all-around results, but it is an excellent example of the difference in performance under static and fatigue loading conditions. While it showed the best results in the static tests, later investigations in fatigue have shown that a specimen of Type C is better than Type E for resisting repeated loads. The narrow point of Type E acts as a stress raiser, due to the concentration of stress. From the standpoint of ease in preparing the material, the butt straps of Type C are the simplest. For wide members, the Type B splice is a good one.

The authors state that the data refer only to stresses produced in the specimens by direct tension loads and that the relative suitability of different types of splices may be affected by a change in the method of loading.

Structures or machine parts subjected to repeated loads require special consideration where fatigue stresses may occur before the yield point is reached. Here "the knowledge of the distribution of stresses in various types of welded connections will help to select the proper detail for each particular case."

In an investigation reported by F. R. Freeman, Esq., before the Institution of Civil Engineers in England, one type of specimen was that shown in the lower left-hand corner of Fig. 36. He stated that, "This type of joint would appear to be very practicable, as the stress tends to be very evenly distributed over the weld and plate with a short length of weld."

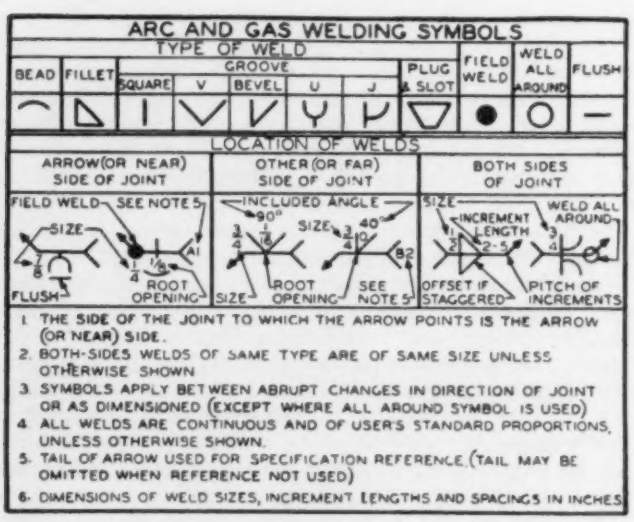
ARC AND GAS WELDING SYMBOLS
TYPE OF WELD
BEAD
FILLET
GROOVE
SQUARE
V
BEVEL
U
J
PLUG & SLOT
FIELD WELD
WELD ALL AROUND
FLUSH
LOCATION OF WELDS
ARROW(OR NEAR) SIDE OF JOINT
OTHER(OR FAR) SIDE OF JOINT
BOTH SIDES OF JOINT
FIELD WELD
SEE NOTE 5
SIZE
FLUSH
ROOT OPENING
INCLUDED ANGLE
90°
40°
SIZE
ROOT OPENING
SEE NOTE 5
INCREMENT LENGTH
WELD ALL AROUND
OFFSET IF STAGGERED
PITCH OF INCREMENTS
1. THE SIDE OF THE JOINT TO WHICH THE ARROW POINTS IS THE ARROW (OR NEAR) SIDE.
2. BOTH-SIDES WELDS OF SAME TYPE ARE OF SAME SIZE UNLESS OTHERWISE SHOWN
3. SYMBOLS APPLY BETWEEN ABRUPT CHANGES IN DIRECTION OF JOINT OR AS DIMENSIONED (EXCEPT WHERE ALL AROUND SYMBOL IS USED)
4. ALL WELDS ARE CONTINUOUS AND OF USER'S STANDARD PROPORTIONS, UNLESS OTHERWISE SHOWN.
5. TAIL OF ARROW USED FOR SPECIFICATION REFERENCE (TAIL MAY BE OMITTED WHEN REFERENCE NOT USED)
6. DIMENSIONS OF WELD SIZES, INCREMENT LENGTHS AND SPACINGS IN INCHES

Fig. 35

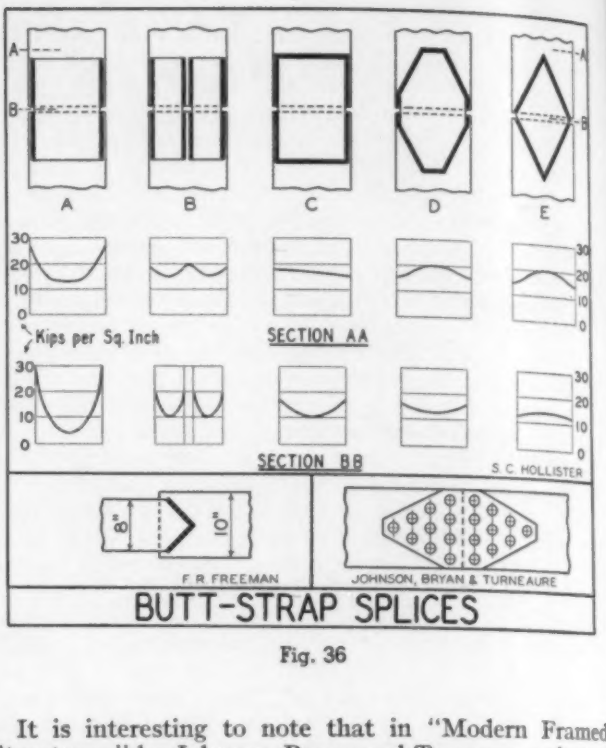

Fig. 36

It is interesting to note that in "Modern Framed Structures," by Johnson, Bryan and Turneare, a riveted splice with tapered splice plates (Fig. 36) is shown to give a better distribution of stress than a square joint.

Plate Girders

The simplest form of welded plate girder is shown in Fig. 37 (*A*). It consists of a web plate, as in the usual riveted plate girder, but the flanges are composed of plates which are welded directly to the web. When it is necessary to increase the flange area it may be done by adding cover plates as shown in Figs. 37 (*B*) and 37 (*C*). It is quite general practice, especially when strengthening existing beams or girders, to make the bottom flange cover wider than the flange or flange plate and the top cover narrower than the flange or flange plate (Fig. 37

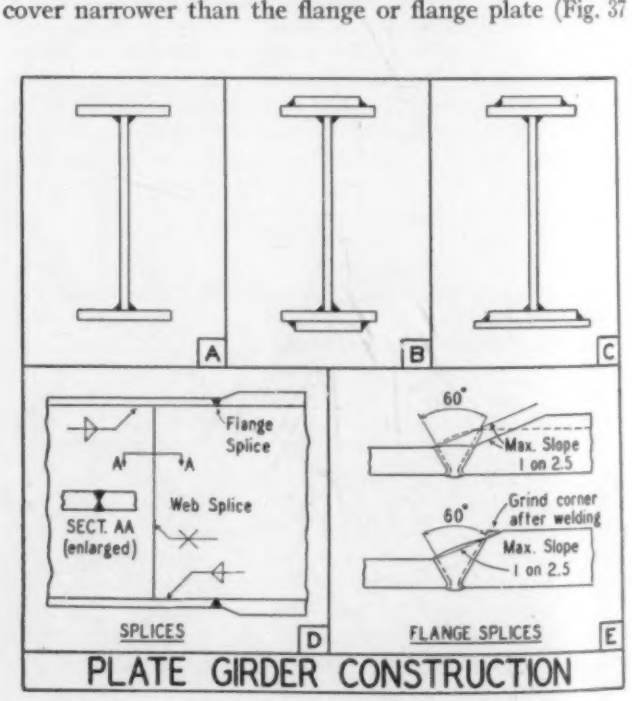

Fig. 37

(*C*)) in order that the welding on the covers may be performed in a downward position.

The A.W.S. Specifications for Welded Highway and Railway Bridges in Art. 229 state that "the number of flange plates shall preferably be limited to one each for each flange" and that "the thickness of this flange plate may be stepped down as required, the abutting ends of the different thicknesses being butt welded." It is further provided that "the splices shall either develop the full effective strength of the material or they shall develop the strength required by the total stresses, calculated on the assumption that all live and impact loads are increased 15%, but in no case shall such strength developed be less than 50% of the effective strength of the material spliced." This arrangement is shown in the flange splices of Fig. 37 (*D*) and the details of the joint in Fig. 37 (*E*) are those proposed in Art. 224. The writer would prefer the form in which the weld surface remains horizontal, entertaining some doubt as to the smoothness of the contour when the attempt is made to give the weld a sloping surface, unless the weld can be positioned, a procedure which would be difficult with long plates. While a butt splice is shown in the sketch of Fig. 37 (*D*) for both flanges, the splicing of the tension flange in this manner should be done with great care, particularly where dynamic or fatigue conditions of loading are to be met in service. It is most essential that every precaution be followed to achieve a smooth merging of the weld into the base metal, to make certain there are no discontinuities within the weld due to lack of penetration or porosity and to chip the back of the weld at the root and reweld from that side.

The shop welding of a plate girder is a matter of importance, if trouble is to be avoided. The flanges should be completed separately, before they are assembled with the web. Likewise, the web should be spliced and welded in advance, if that is necessary, using a complete penetration groove weld. It is usual practice to have two welders working simultaneously on opposite sides of the web, so as to preserve symmetry of operations and thus minimize distortion. In order to avoid locked-up stresses of a serious character due to the longitudinal contraction of the flange-to-web welds, the welding should begin at the center of the girder and proceed outwardly toward the ends, leaving the ends free to move as the welds contract.

Stiffeners can be bars attached to the web with continuous or intermittent welds, designed to transmit the stresses. Tees and other shapes are also used. Examples of the use of bars are given in Fig. 38. The stiffeners of section *AA* are shown welded to the top and bottom flanges, but at points where the flanges are

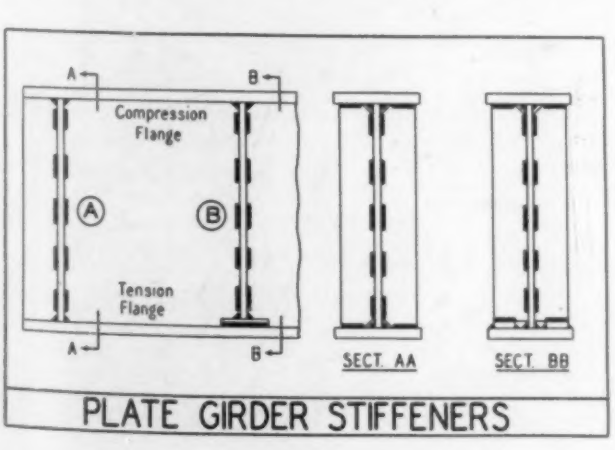

Fig. 38

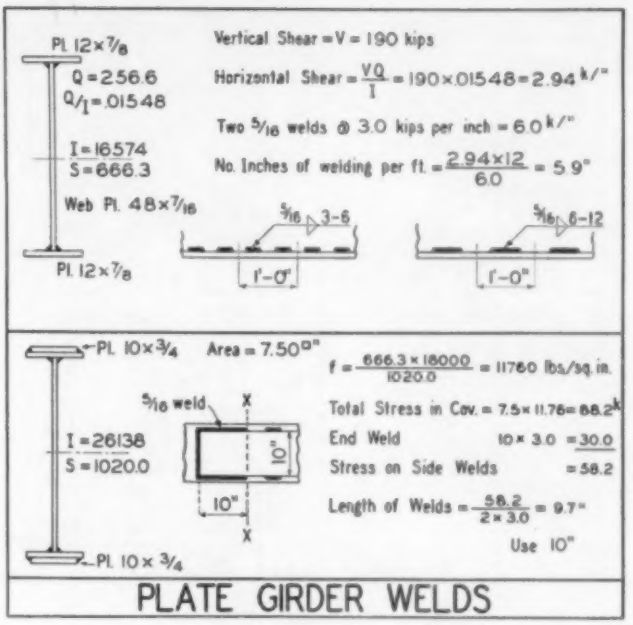

Fig. 39

carrying any considerable amount of stress, this is bad practice on the tension flange. The reason is at once clear when we consider the stress paths and the notch effect introduced by the fillet welds transverse to the stress. The placing of fillet welds on any tension member across the line of stress should be avoided. At the ends of a girder the stress in the flanges is practically zero and there it is permissible and may be desirable to weld the stiffener to the tension flange (or bottom flange) in order to lend support to the stiffener in transmitting the end reaction.

It will be noted that the stiffeners are notched to clear the longitudinal welds. This is desirable for several reasons. In the first place, the stiffeners are welded in place after the web and flanges have been welded and therefore the corners of the stiffeners have to be trimmed for clearance. Furthermore it is a wise practice to avoid welding in a corner or pocket from several directions, and by notching the stiffeners and removing a portion of them in the region adjacent to the longitudinal welds any chance of welding the stiffeners close to these welds is made impossible.

There are times when it may be desirable to attach the stiffeners to the tension flange in regions where the stress is considerable. In such cases, a shim or filler or short bar may be placed under the ends of the stiffeners as shown in section *BB*. Then the stiffener is welded with transverse welds to this bar and the bar is in turn welded with longitudinal welds to the tension flange—thus avoiding transverse welds on the flange plate itself. Sometimes the bar is driven to a fit and welded only to the stiffener.

The calculations for the welding on a plate girder are given in Fig. 39. The properties of the cross section have been found by the usual methods and are as follows:

Moment of inertia, $I = 16{,}574$
Section modulus, $S = 666.3$
Statical moment of flange, $Q = 256.6$

Assume the vertical shear, V, to be equal to 190 kips. From the well-known formula the horizontal shear between the web and the flange plates $VQ/I = 2.94$ kips per inch or 35.3 kips per foot of girder. Two $^5/_{16}$-in. welds at 3.0 kips are good for 6.0 kips per inch. Hence, in 1-ft. length of girder there are required 35.3/6.0 =

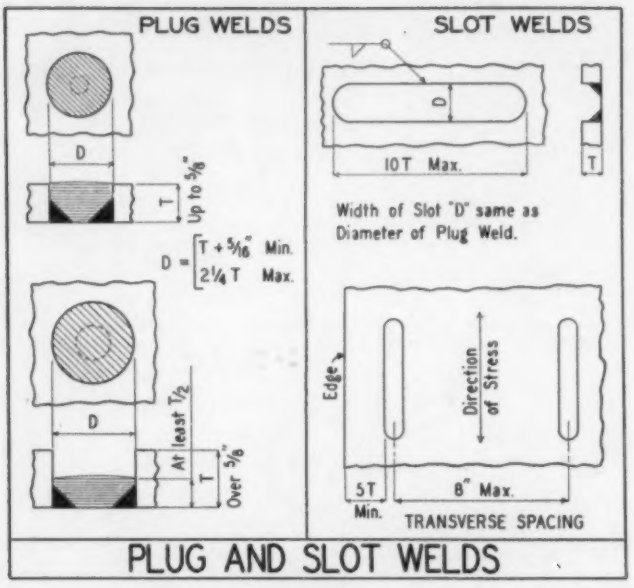

Fig. 40

5.9 in. of welding, or practically 6 in. Two alternative arrangements of the welding are shown. The 3-in. welds at 6-in. centers give a more uniform distribution of shear on the web. More about intermittent welding will be discussed in a later section dealing with "Fatigue."

At the ends of any girder it is desirable to make the welding continuous for a reasonable distance even though the design calculations do not require it. Where the girder will be exposed to the weather it is essential to seal the joints with continuous welds, increasing the size only as required by the stress to be transmitted.

As in riveted construction, covers should extend past the point at which they are required in order to develop their stress. Assume 10 x $^3/_4$ covers added to the plate girder in the upper half of Fig. 39. The fiber stress in the original girder flange will be assumed as 18,000 psi. where the covers are needed. The necessary calculations are shown in the sketch with a result which calls for the covers to extend 10 in. beyond the theoretical point designated by the line *XX*.

The design of stiffeners and their spacing can follow the usual structural practices. The welding on bearing stiffeners should be sufficient to transmit the load from the stiffeners to the web plate.

Plug and Slot Welds

Where cover plates are added to a member it is sometimes desirable to connect them with additional slot or plug welds to prevent the plates from buckling transversely or perhaps to supplement the edge welds where space is not sufficient to transmit all the stress through the edge welds. The requirements as to dimensions are shown in Fig. 40. The term "filled" applies to both plug and slot welds in material up to $^5/_8$ in. in thickness when the hole is filled with metal, or at least to one-half the thickness of material over $^5/_8$-in. thick.

The effective area of fillet welded plug and slot welds is considered to be the length of the center line of the weld through the center of the throat times the effective throat thickness. Where the toes of the fillets overlap in the center of the hole or slot, the effective area shall not exceed the nominal cross-section area of the hole or slot.

If filled plug and slot welds are not to crack from shrinkage stresses a definite technique must be followed, as detailed in an article "Shear Tests of Plug and Slot Welds" by C. E. Loos and F. H. Dill, published in The Welding Journal of February 1940. Quoting a paragraph from that paper:

"The arc was carried around the root of the joint and then weaved along a spiral path to the center of the hole, depositing and fusing a layer of weld metal in the root and bottom of the joint. The arc was then carried to the periphery of the hole and the procedure repeated, depositing and fusing successive layers to fill the hole to the depth required. The slag covering the weld metal was kept molten, or nearly so, until the weld was finished. If the arc was broken, except briefly for changing electrodes, the slag was allowed to cool and was completely removed before restarting the weld."

The conclusions of Loos and Dill are these:

1. Plug welds may be stressed in shear at the working stresses ordinarily allowed for fillet welds. The nominal diameter or size of the hole in which the weld is made should be used in all calculations of its working stresses or load-carrying capacity.
2. Plug welds in plates up to $^5/_8$ in. thickness should be made flush with the plate surface. Holes in thicker plates may be filled only one-half of their depth, if only strength and economy govern their design. Plug welds in plates over $^3/_4$ in. thick are not usually economical.
3. The test of slot welds showed that their characteristics are essentially the same as those of plug welds.

Trusses

The design of welded trusses has departed from riveted practice by eliminating the gusset plate wherever possible and joining the members directly to one another. One development of this new procedure has brought about the use of split-beam chord sections, the beam webs acting as the gussets to which the truss members are welded. A typical joint is shown in Fig. 41. The calculations for the lengths of the welds on the tension diagonal are given and follow the principles set forth in Fig. 31. The center of gravity lines of the angles meet at a common point in order to avoid eccentricity. The size of the beam selected for the chord section depends

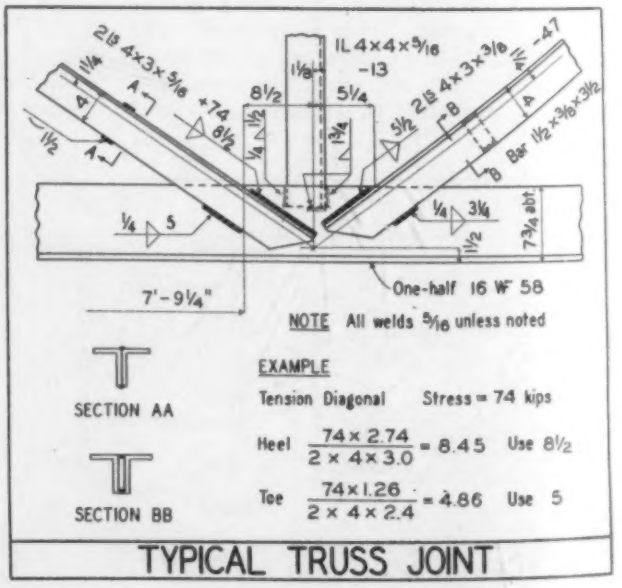
2 ⌊s 4x3x5/16 +74
1L 4x4x5/16 -13
2 ⌊s 4x3x3/8 -47
Bar 1/2x3/16x3 1/2
8 1/2
5 1/4
7'-9 1/4"
One-half 16 WF 58
NOTE All welds 5/16 unless noted
SECTION AA
SECTION BB
EXAMPLE
Tension Diagonal Stress = 74 kips
Heel (74 x 2.74)/(2 x 4 x 3.0) = 8.45 Use 8 1/2
Toe (74 x 1.26)/(2 x 4 x 2.4) = 4.86 Use 5
TYPICAL TRUSS JOINT

Fig. 41

upon two things; first, it must have the required area, and second, the stem formed by splitting the beam on the center line of the web must be wide enough to accommodate the welding required on the diagonal members.

The thickness of the stem should not be less than one-twenty-fourth the distance from the beam fillet to the edge of the stem. When double members are connected with welds on opposite sides it is necessary to examine the stem for shear. The following table gives suggested minimum thicknesses of stem for varying sizes of welds:

Weld Size, In.	Min. Thickness, In.
1/4	0.22
5/16	0.27
3/8	0.33
1/2	0.43
5/8	0.54

The 16 WF 58 has a web thickness of 0.407 in. and the stem is, therefore, amply strong against the shear from the 5/16-in. welds on opposite sides which require a minimum thickness of 0.27 in. The distance from the fillet to the edge equals 7.75 − 1.32 = 6.43 in. and one-twenty-fourth this distance = 0.27 in., the minimum thickness which should be used. Again the 16 WF 58 satisfies the limiting conditions.

Section *AA* illustrates one method of tying double-angle tension members together. Compression members should have fillers (equivalent to washers in riveted work) as shown in section *BB*, and fillers may likewise be used in tension members.

Figure 42 shows the important joints of an ordinary Fink-type truss. Because of its size the truss was shipped in sections with field splices at joints *B* and *C* and at the bottom of the vertical center members. The truss was assembled on the ground and the splices welded before erection. Split beams were used for the top and bottom chords and the web members were angles. It will be noted that the bottom chord between joints *A* and *B* has no holes punched in it so that the gross area can be used in tension. The holes for the erection bolts connecting the truss to the top of the column were placed in a plate welded to the under side

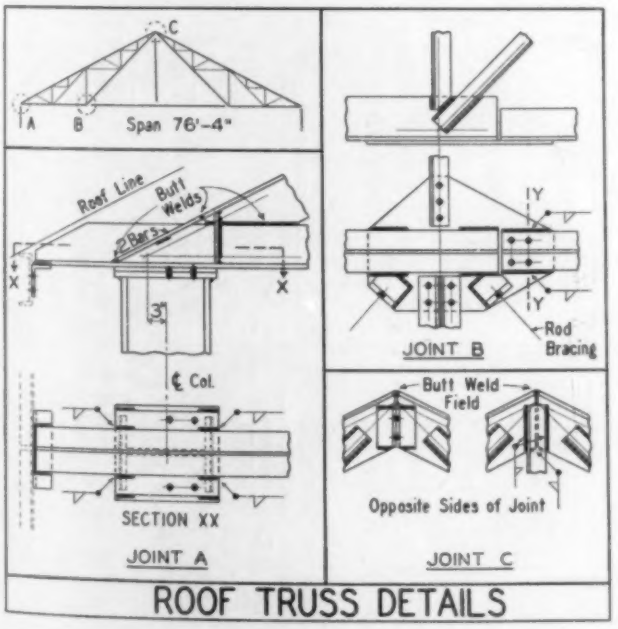

Fig. 42

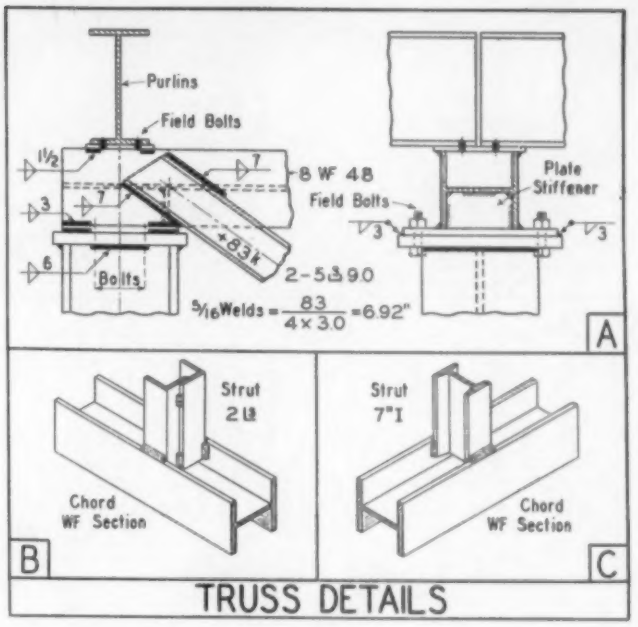

Fig. 43

of the bottom chord. A brief examination of the details of joint *A* will show how the stresses were transferred.

Joint *B* shows the bracing gusset plate which was shop welded to the bottom chord of the half-truss. The center section of the chord had no shop-welding operations so it was decided to punch the erection assembly holes directly in this member. The first holes on section *YY* were located far enough back from the ends of the field welds so that the stress on the net section would be reduced below the working value by the amount of the stress removed from the member by the welds before the section was reached.

The peak of the truss is shown in joint *C*. The assembling of the truss sections was facilitated at this point by shop welding a short angle to each half of the truss with holes to match. The two field bolts drew the parts quickly into position and alignment. Another point of interest is the splicing of the chord flanges by a field butt weld. The erector reported that this splice added a great deal of lateral stiffness to the joint during handling and erection of the truss. The stems were spliced by means of the vertical angle members, the attached leg serving practically as a splice plate. The toes of the assembly angles were welded together. In order to guard against excessive stress in the edge of the stem due to the truss deflection, the edges were welded to the vertical angle as shown.

Figure 43 (*A*) is a joint from a heavy truss of the Pratt type in which the chords were wide-flange sections. Here the flanges of the WF sections performed the function of gusset plates. The diagonals were channels and the compression verticals were either angles or 7-in. I-beams as shown in Figs. 43 (*B*) and 43 (*C*). The welds on the channels were calculated according to the method explained with Fig. 30. A small plate stiffener was welded between the WF flanges to add support to the flanges and assist in transferring the reaction to the column. Holes for erection were kept in the detail pieces welded to the main material. This avoided extra handling of the heavy chord sections to the punches in the shop, and illustrates the points previously emphasized that the combination of welding and punching on the same main members is not desirable.

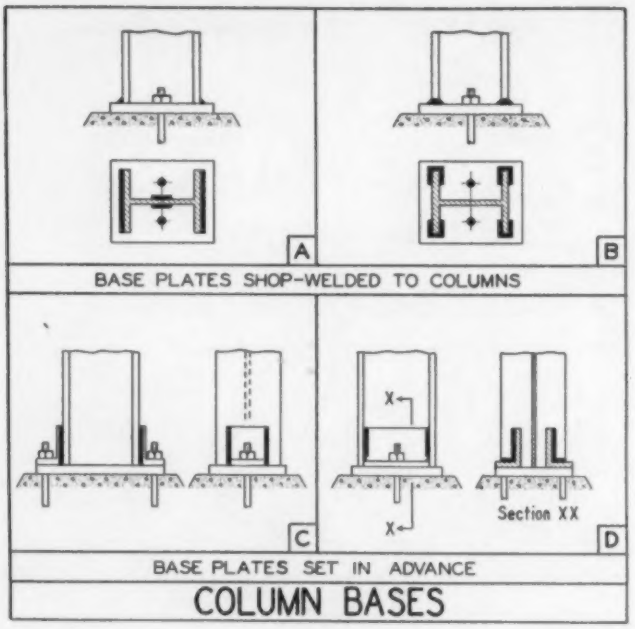

Fig. 44

Column Bases

The simpler forms of column bases may be divided into two classes—one in which the base plate is shop welded to the column and the other in which the base plate is shipped separately and set in place on the masonry before the column is erected Examples of both types are shown in Fig. 44. *A* and *B* show two typical arrangements of the shop welding. The first is the simpler from a fabrication standpoint but the latter is a more effective distribution of the welds. In the first type one of the lines of weld is sometimes placed on the inside of the column flange to save the operation of turning the column over on the skids in order to bring the weld into position. A ready rule for the amount of welding around each flange toe of base *B* is to make the total length equal to one-half the width of the flange. This makes the amount of welding the same on both bases. Web welds, as shown in *A*, are not necessary on small columns but are desirable on wide columns such as occur in industrial mill buildings.

Bases *C* and *D* illustrate two methods of shop welding base angles to a column when the base plate is shipped loose and set in advance. The choice of details depends on the location of the anchor bolts. In *D* the angles are set out from the web at least far enough to clear the column fillets and are welded to the column flanges. In both types the angles should be set slightly (perhaps $^1/_{32}$ in.) back from the finished end of the column to insure against the possibility of their bearing on the base plate and so throwing stress on the welds.

A more important class of column bases, from the point of view of design, is that in which provision must be made for uplift. These are the bases which require the familiar "boot" construction often found on such structures as mill buildings and crane runways. Figure 45 shows several forms of welded construction.

In base *A* 5 x 5 x $^3/_8$ angles were welded to the column shaft and the anchor bolts located so as to have about $^1/_8$-in. clearance from the inside faces of the angles. The anchorages on base *B* were made from pieces of 3-in. Double Extra Strong Pipe, gas welded to the edges of the column flanges. It will be noted that in both types the angles and the pipes are set back from the finished end of the column to avoid any possibility of the direct column load being carried on the boots.

Base *C* is still another type in which bar stiffeners in pairs are welded to the column with small bearing plates welded to their upper ends upon which the nuts rest As in bases *A* and *B*, the usual practice was followed of setting the base plate in advance of the steel erection.

Without going into the design of a structure let it be assumed that the calculated uplift on one of the anchor bolts in base *C* is 90 kips. The essential features surrounding the design of this anchorage will be illustrated in the following calculation:

At 18,000 + 33$^1/_3$% = 24,000 psi.
Required area of bolt = 90/24 = 3.75 sq. in.
Net area of 2$^1/_2$-in. bolt = 3.72 sq. in.

Since the long diameter of a hexagonal nut is 4$^1/_2$ in. the center of the bolt must be out from the column face at least 2$^1/_4$ in. + $^1/_2$ in. for clearance. In order to make the center-to-center distance of anchor bolts an even figure let the bolts be placed 3 in. from the face of the column.

The length of the stiffeners is determined by the amount of welding required to transfer the load of 90 kips from the column to the bolt. Since there are four lines of vertical welds, each weld will carry 90/4 = 22.5 kips. The chart in Fig. 33 will give the answer when the coefficient is calculated.

For $^5/_{16}$-in. welds, $R = 3.0$ kips per inch
$3.0 = 22.5 \times K$
$K = 0.133$

For this value of K and an eccentricity of $a = 3$ in., it is found that 13 in. of welding is required.

The top bearing plate must be at least 5$^1/_2$ in. wide in order to provide a seat for the nut and the stiffeners will be made of this width. With this width fixed, the minimum clear distance between stiffeners can be determined as shown in the sketch. Although 3$^3/_4$ in. is indicated, it would be better to use 4 in. as this provides slightly more clearance for the electrode and is an easier measurement for the shop.

In all bases in which uplift is a factor special attention should be given to the horizontal shear on the column

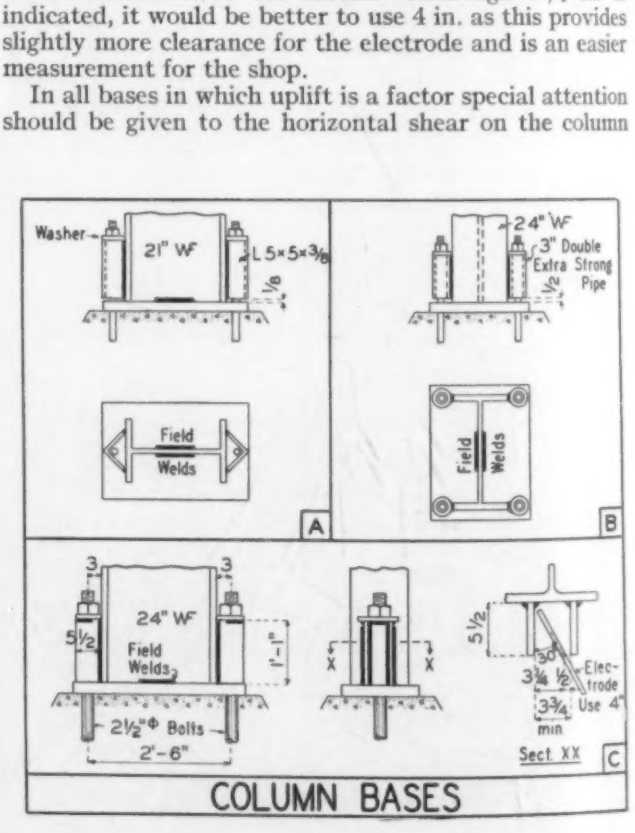

Fig. 45

and the shaft welded to the base plate with sufficient welding to transmit this shear.

Column Splices

In tier buildings where the column splices serve more particularly to hold the abutting sections in line and not primarily to transmit stresses, it will be sufficient to substitute welding for riveting directly. For erection purposes it is necessary to provide bolt holes so that the members can be held in place until the field welding is done. Figure 46 shows the splices which were used in the nineteen-story office building of the Dallas Light & Power Company in Dallas, Texas. (NOTE: This sketch and those in Figs. 47 and 48 have been taken directly from the original paper without redrafting them for the present symbols. *F.W.* designates field weld and *S.W.* shop weld.)

These are excellent examples of what might be considered good practice. If any suggestion of change were to be made, it might have been possible to shorten the filler plates 3 in. and place 3 in. of horizontal welds on each side of the ends of the fillers. The balance between the welds on the fillers and the upper half of the splice plates is worthy of comment. In splice *B* 6 in. of $^{3}/_{8}$-in. welds at 3000 lb. per inch are worth 18,000 lb. and 9 in. of $^{1}/_{4}$-in. welds at 2000 psi. are also worth 18,000 lb. In splice *C*, 9 in. of $^{3}/_{8}$-in. welds at 3000 lb. per inch are worth 27,000 lb. This force is acting $1^{7}/_{8}$ in. from the line of welds on the edges of the filler plate. By referring to the chart, Fig. 33, and calculating the value of *K* we can determine the required length of welding.

$3.0 = 27 \times K$
$K = {}^{3}/_{27} = 0.111$
For $a = 1^{7}/_{8}$ and $K = 0.111$.
Required $L = 12$ in.

In the above calculations it should be borne in mind that this building was built more than ten years ago in the era of bare wire electrodes, when the working unit stress in shear was 11,300 psi. instead of the present value of 13,600.

Another type of splice for tier buildings is that illustrated in Fig. 46 (*D*), the distinctive feature of which is that it avoids fillers and the punching of holes in the

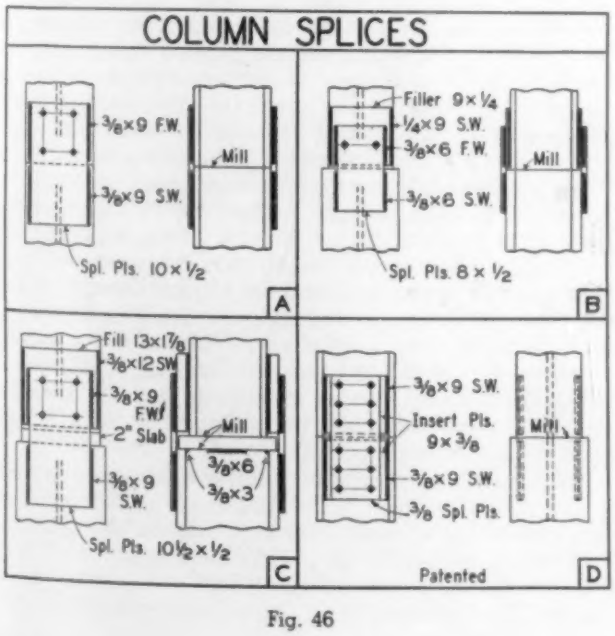

Fig. 46

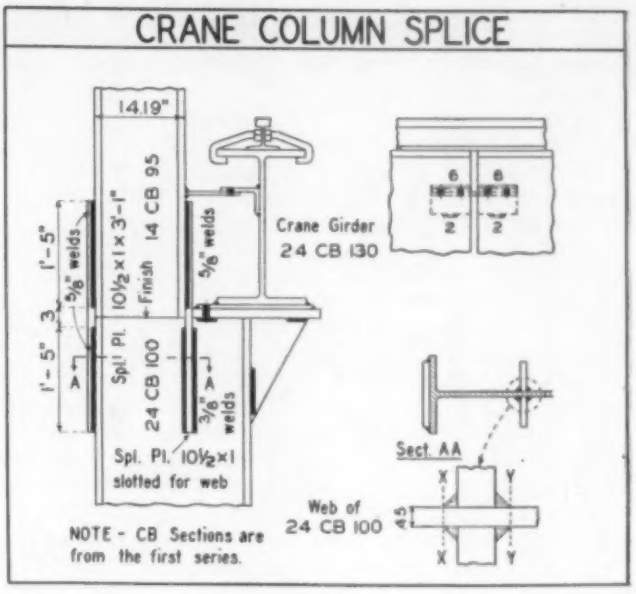

Fig. 47

flanges of the column. The insert plates are shop welded to the column flanges and contain the holes for the field bolts. The splice plate may be welded in the field to the insert plates, although in many instances, as definitely prescribed by the A.I.S.C. Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, the field connection may be permanently bolted.

Another class of column splices is to be found in crane columns of mill buildings in which bending moments enter into the problem. A typical example is shown in Fig. 47. The building in which this occurred was designed on the basis of 16,000 psi. in tension on the steel, in accordance with the governing building ordinances. On this same basis, the splice was calculated for the allowable bending moment on the upper 14 CB 95 section.

14 CB 95 $S = 150.5$
$M = 16 \times 150.5 = 2410$ in. kips

$$\text{Flange stress} = \frac{2410}{14.19} = 170 \text{ kips}$$

Outside Splice plate

Required area = 170/16 = 10.6 sq. in.
Plate $10^{1}/_{2} \times 1 = 10.5$ sq. in.

In order to avoid an excessive length of splice it was decided to use $^{5}/_{8}$-in. welds which have a value of 5000 lb. per linear inch.

Required length of welds = 170/(2 x 5) = 17.0 in.

Inside Splice Plate

The same size, $10^{1}/_{2} \times 1$, was used and slotted for the web of the lower section, 24 CB 100. The welding on the upper half was made the same as the outside plate. On the lower half, the shear on sections *XX* and *YY* of the 24 CB web was investigated on the basis of 10,000 psi. To resist the flange stress of 170 kips required a length = 170/(2 x 10 x 0.45) = 18.9 in. As this was practically the length of the outside splice plate below the splice line it was made the same. Four $^{3}/_{8}$-in. welds 17 in. long at 3000 lb. per inch have a total value of 4 x 3 x 17 = 204 kips which is in excess of the requirements of 170 kips.

Figure 48 is a similar splice in which the inside flange of the upper section is spliced by means of two plates,

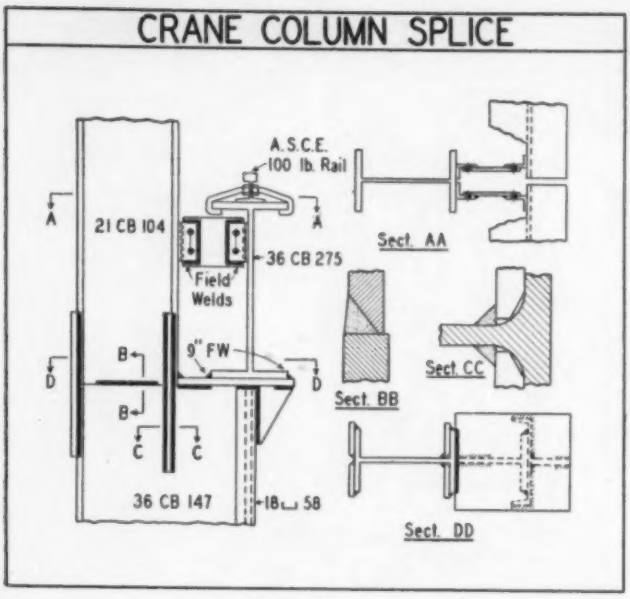

Fig. 48

one on either side of the web. The edges were beveled so as to clear most of the fillet and an oversized weld used to make up for the gap. A horizontal weld was placed along the joint line to carry the horizontal shear. The outside flange splice was made with two plates in order to shorten the over-all length of the splice by using four lines of weld (see Fig. 36, splice *B*).

Beam Connections

It is probably safe to say that beam connections have been the most difficult problem confronting the engineer in the design of welded structures. During the past ten years the problem has received a very considerable amount of study and research—much of it being done at Lehigh University under the sponsorship of the Structural Steel Welding Committee and the Welding Research Committee of the AMERICAN WELDING SOCIETY. The papers resulting from these investigations have been published in THE WELDING JOURNAL of the SOCIETY and are listed in the bibliography at the end of this paper.

In approaching the subject it may be helpful to review very briefly the action of a beam when under load. Assume a beam to be simply supported (free-ended) as shown by the full lines of Fig. 49. Under the action of vertical loads, the beam will deflect an amount, Δ, to the position indicated by the dotted lines. The ends of the beam will be inclined by the amount of the angle, θ, and the top flange displaced a distance, b, in relation to the bottom flange. The table gives the formulas for these quantities for three common loadings. Two sets of formulas are given: one when the maximum fiber stress in bending is 20,000 psi. the other when the center deflection equals $^1/_{360}$ of the span.

The function of a beam connection is to transfer the end reaction or shear to the supporting member, be it a column, plate girder or another member. Figure 50

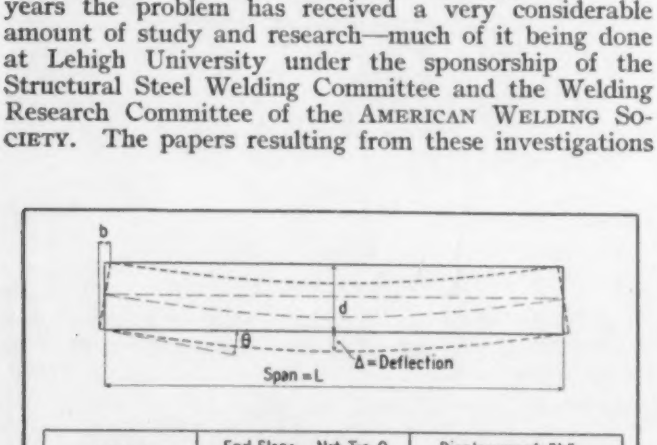

Fig. 49

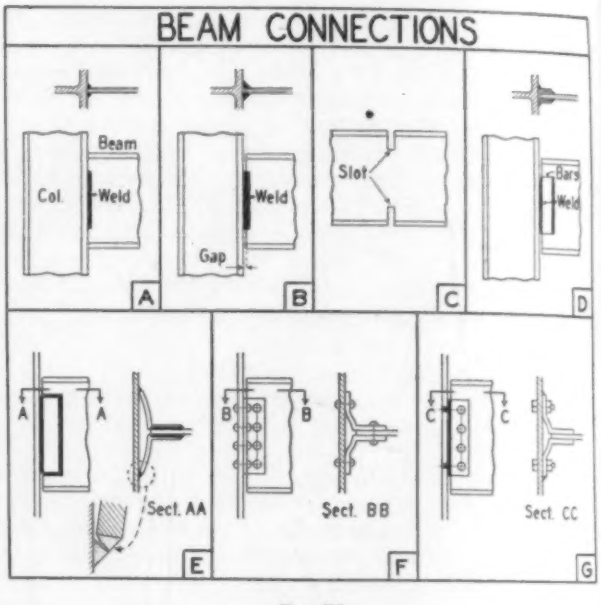

Fig. 50

(*A*) illustrates one type of connection which has undoubtedly suggested itself to many on first thought. The end of the beam is butted against the column and the web welded directly for the shear. Apart from the question of the adequacy of the connection there is a practical objection to this method. The welding might span a gap of $^1/_8$ in. as in Fig. 50 (*B*), (Art. 603 (*a*) of A.W.S. Bridge Specifications sets a maximum of $^3/_{16}$ in. for such a gap) but even with this tolerance it would be extremely difficult to keep within that limit unless the beam were finished to length—an expensive operation. Even then the erector would have to contend with overrun on columns, inaccuracies of location due to play in anchor boltholes and other causes readily evident to those familiar with field work. This difficulty can be overcome by the use of bars on either side of the web as in Fig. 50 (*D*). However, some such device as an erection seat would have to be provided to support the beam until welded.

The connection should be analyzed for strength. As an example take an 18 WF 47 beam on a 30-ft. span for which the total uniform load is 36,600 lb. at a bending fiber stress of 20,000 psi. The end reaction is then 18,300 lb. which requires 18,300/3000 = 6.1 in. of $^5/_{16}$-in. fillet weld, or practically $3^1/_4$ in. on each side of the web.

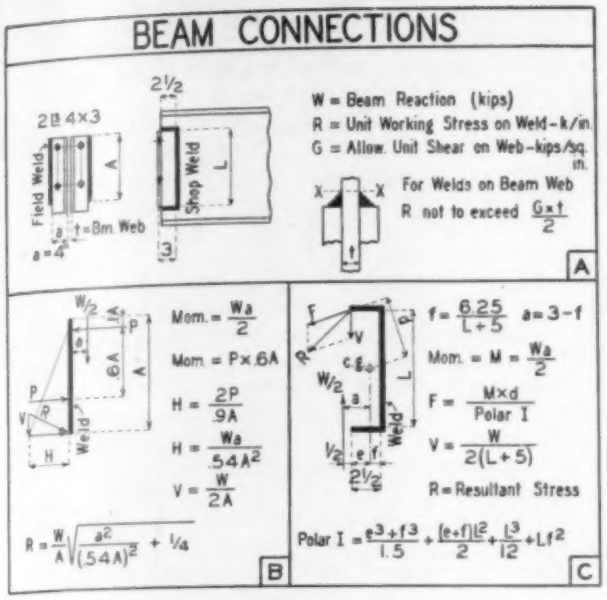

Fig. 51

Assume that we have the condition in Fig. 50 (*B*) and the welds increased in size in accordance with Fig. 34 (*D*). From the table of Fig. 49 the end slope of the beam (considering it to be simply supported) is found to be 0.00925. If the rotation takes place about the center line of the welds, the displacement at the top of the welds will be equal to $0.00925 \times 1.5 = 0.0139$ in. Recalling that structural steel at the yield point stress increases in length about 0.001 in. per inch, it is apparent how seriously overstressed the weld and the adjacent beam web would be. The welds tend to fix the end of the beam by their resistance to the deformation, but they are wholly inadequate to do so. The condition at the connection is much the same as would exist in Fig. 50 (*C*), were slots to be cut to a depth such as to leave a width of web equal to the length of the welds.

There is another side to the strength of such a connection that should be considered. From the discussion in the section dealing with "Stress Paths," it must be clear that the tops of the fillet welds on the web connection are particularly liable to stress concentrations. The occurrence of stress raisers at points where the normal stresses are known to be in the region of the yield stress is highly undesirable and even dangerous when used alone.

It has been suggested that the welds be made heavy and so force the deformation to take place in the web. This seems, at best, a rather doubtful expedient. The reasonable and safe procedure is to avoid this type of web connection unless the beam flanges are sufficiently held as to relieve a substantial part of the end rotation moment otherwise carried entirely by the vertical web welds. The presence of repeated load conditions would rule the connection out altogether.

A type of connection which has been used is shown in Fig. 50 (*E*). It is an attempt to utilize the flexibility of the outstanding legs of the connection angles to provide for the turning motion of the end of the beam. (All structural engineers recognize that such an action takes place in the ordinary riveted connection of Fig. 50 (*F*).) This welded connection should be used with caution, especially when the angles are 6 in. or more in length. The enlarged view of the weld will illustrate the fact that the deformation of the angle tends to produce tension at the root of the weld, a condition which should be avoided, especially where variable stresses are frequent in cycle.

This same type of connection can be utilized with much greater safety if erection bolts are provided near the top of the angles and left permanently in the structure. In the 15-story building of the Edison Electric Illuminating Company in Boston, a typical framing connection is that shown in Fig. 50 (*G*). Although shop riveted, all the field connections were welded. The erection bolts were permanent and serve to confine the bending stresses to the angles, while the welds carry only the vertical shearing loads. A similar all-welded connection was used on the 19-story building for the Dallas Power and Light Company in Dallas, Texas.

The design calculations for the welding on such a connection are explained in Fig. 51. The problem is to find the lengths for the shop and field welds. The shop weld for one angle is shown in Fig. 51 (*C*). The first step is to locate the center of gravity of the weld and then compute its polar moment of inertia. (In all calculations of this kind the writer has found it simpler to treat the weld as a line and then use a working stress for the weld in terms of pounds per inch. This avoids the introduction of the throat dimension and simplifies changes in size of the weld.) The essential steps are indicated for obtaining the components F and V of the resultant stress, R, on the weld.

It is evident that the web thickness should enter the problem, just as in riveted construction. In Fig. 51 (*A*), imagine the section XX to be taken through the web just above the horizontal welds on the top ends of the connection angles. If the working strength of the two

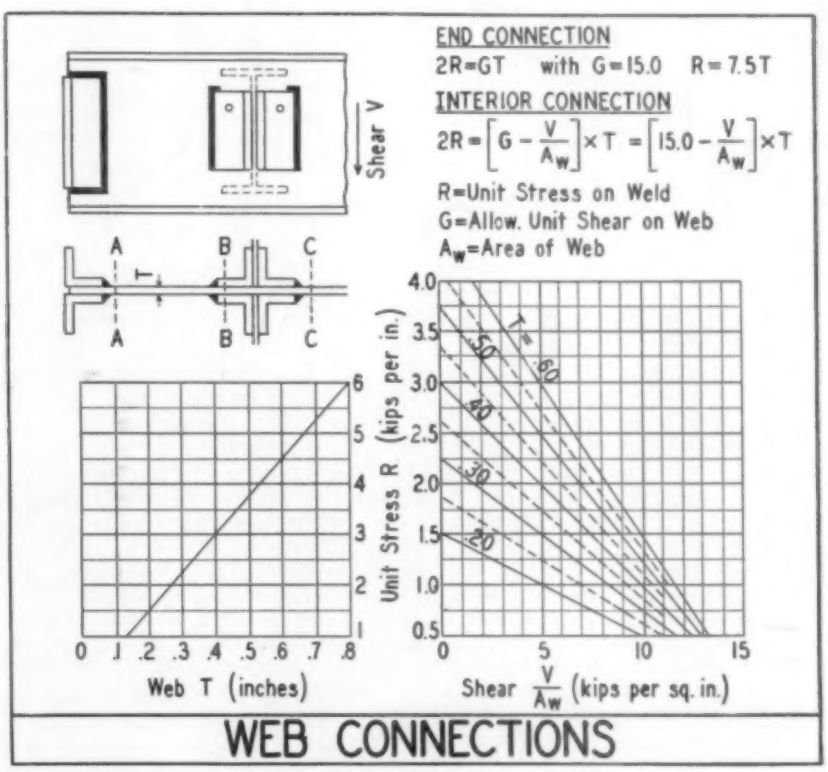

Fig. 52

welds is limited by the shearing stress on the web, then $R = G \times t/2$ for each weld. The A.I.S.C. Specifications provide for G = 13,000 psi. on the gross area of the beam or girder web. This value is based upon certain considerations for web buckling. Since the connection angles are not connected to the beam for the full depth of the web and because the web itself is not permitted to carry more than 13,000 psi. in shear, regardless of the connection, it appears reasonable to use a value of G equal to 15,000 psi. in determining the limiting values of the unit stress on the welds.

Figure 52 contains two simple charts for determining R, based upon G = 15,000 psi., or 15.0 kips per square inch. The left-hand chart covers the case of end connections, discussed in the previous paragraph. For example, a beam with a web thickness of T = 0.40 would limit the weld strength to R = 3.0 kips per inch. From Column A of Fig. 28, we find that $^5/_{16}$-in. welds are limiting size. Of course larger sized welds may be used, but the length must still be that determined on the basis of R = 3.0 kips per inch, if the shear is to be properly distributed over the beam web.

The right-hand chart of Fig. 52 treats the condition that prevails where beams frame to opposite sides of a girder-beam web. Here the web is already carrying a certain amount of shear, V, which gives a unit shear of V/A_w, where V = total shear on the section in kips, and A_w = gross area of the web in square inches. Evidently the amount of shear added to the web by the welds must be such as not to increase the total unit shear in the web opposite the welds above G = 15.0 kips per square inch. The formula for this condition is given above the chart and is the basis upon which the chart was constructed. If V is the total shear existing at section CC (due regard being paid as to whether the shear is positive or negative) then the shear on section BB will be increased by one-half the combined reactions of the two beams framing to the girder beam. The limiting values of R for the welds should be determined for section BB. Assume V/A_w = 2.5 kips per square inch and that the web T = 0.40 in., from the chart, R = 2.5 kips per inch, thus limiting the weld size to $^1/_4$ in. for design purposes, although larger welds may be actually placed on the connection.

Returning to Fig. 51 (B), there is shown the calculations for the field welds. Some designers calculate these welds as being only in direct shear but this method does not take account of the eccentricity of the load $W/2$ with respect to the weld. From the sketch in the upper left-hand corner of Fig. 51 (A) it will be seen that when the load is applied along the center line of the beam web, the angles tend to twist, pressing together at the top and separating at their lower ends. The mutual reaction at the top may be designated by the force, P. Its point of application is indeterminate but a reasonable assumption would be a distance from the top of the angles equal to $^1/_{10}$ of A, the length of the individual welds. Assuming a triangular distribution of horizontal stress on the weld, the resultant horizontal force must equal P and its point of application is $^2/_3$ of $0.9A$ below the upper force, P. The value of P can be found by equating the moment of the horizontal couple to the moment of the load. The intensity of the horizontal stress, H, per inch is readily calculated when P has been found. The vertical stress per inch equals the load divided by the weld length. The resultant stress, R, is found by combining the vertical and horizontal stresses.

When beams frame opposite on a girder beam it is necessary to be certain that the girder beam web is of sufficient thickness to carry the shearing stresses. Reference should be made to the chart of Fig. 52 and the discussion of its use.

It is evident that trial calculations to solve this problem would involve a very considerable amount of time and labor. The diagram of Fig. 53 was prepared to facilitate this operation. By reference to the sketch it will be seen that the field welds are carried over the top edges of the angles for $^1/_2$ in. The writer believes this to be a desirable precaution. This additional length is not included in the calculations.

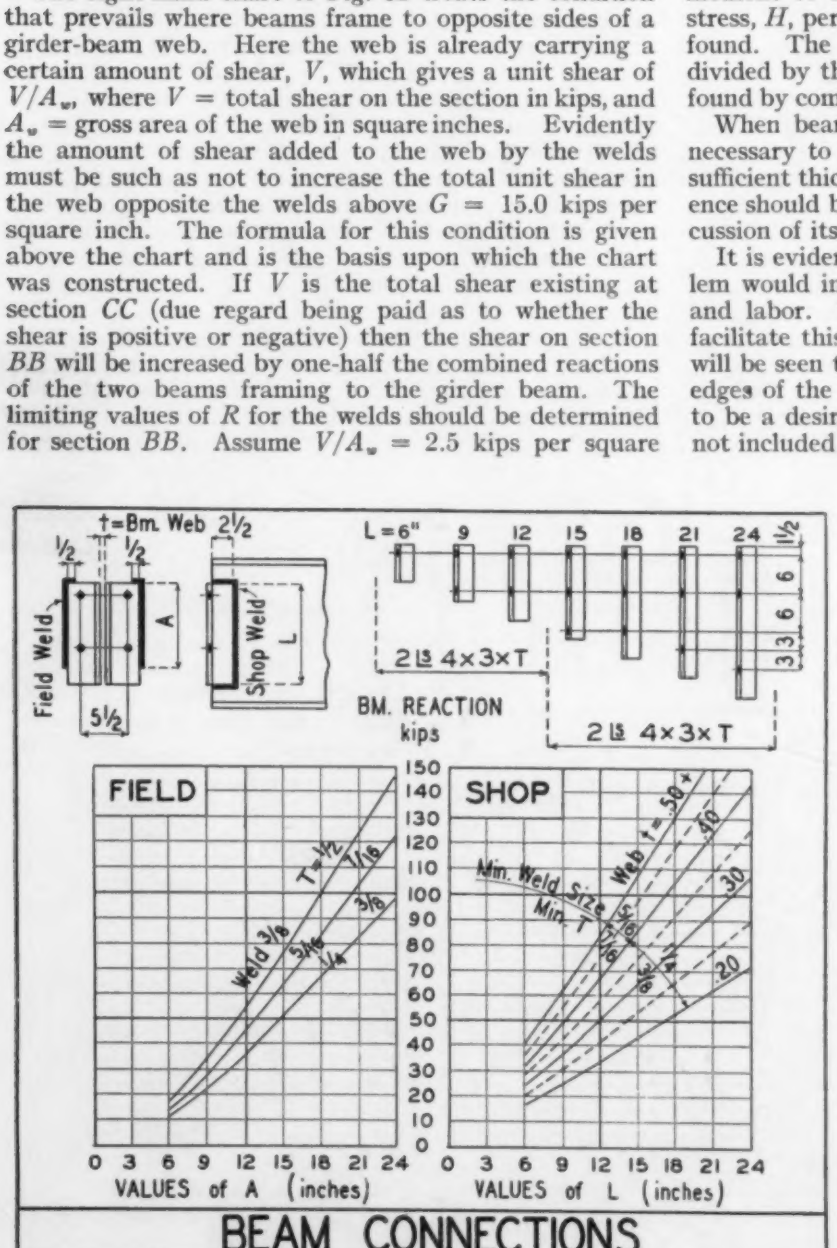

Fig. 53

The use of the charts can be illustrated with a typical example. Assume a 21 WF 59 beam with a reaction of 40 kips. The web thickness of this beam is 0.390 in. Entering the chart for "Shop" welds at the vertical ordinate for 40 kips, move horizontally to the diagonal line (to be interpolated on the chart) for web thickness, t = 0.390 in. The horizontal abscissa is found to be slightly more than $7^1/_2$ in., which is the required length, L, for the shop welds. The intersection falls within the range of $^1/_4$-in. welds and a minimum thickness of $^3/_8$ in. for the angles.

The length of the field welds may be found in a similar manner from the left-hand chart:

A = 13 in. for $^1/_4$-in. welds or 11 in. for $^5/_{16}$-in. welds

Evidently, the field welds are the determining factor in the final length of the connection angles.

A moment's thought will show that an unlimited number of lengths of angles might result from the calculations for each individual beam on a whole job. The writer suggests standard lengths and sizes as shown in the upper right-hand corner of Fig. 53. With the use of these standard angles, the final

connection for the 21 WF 59 beam would be 2Ls 4 x 3 x $^{7}/_{16}$ x 12 in. long with at least $^{5}/_{16}$-in. field welds and $^{1}/_{4}$-in. shop welds. The minimum weld lengths of A = 11 in. and $L = 7^{1}/_{2}$ in., must remain the same even if larger welds are used. In practice, the shop welds would be made for the full length of the angles, L = 12 in.

When two such beams frame opposite, for example, on a 30 WF 116 we have to consider the conditions with respect to the web of the girder beam. Let it be assumed that the two 21-in. beams are represented by the opposite-framing beams of the sketch in Fig. 52 and that the total shear on section CC is -10 kips; i.e., that the portion of the beam to the left of the section tends to slide downward past the right-hand portion under a shearing force of 10 kips. The 30 WF 116 beam has a depth of 30.00 in. and a web thickness of 0.564 in. Hence the area of the web, A_w = 16.92 sq. in. The shear, V, on section BB equals $-10 + 80/2 = +30$. Then V/A_w = 30/16.92 = 1.78 kips per square inch. Entering the right-hand chart of Fig. 52 at this value of V/A_w, follow vertically to the diagonal line (interpolated) for T = 0.564 and read the value of the allowable unit stress, R, for the welds, which is about 3.75 kips per inch. This is the allowable stress for welds between $^{3}/_{8}$ and $^{7}/_{16}$ in. in size (see Column A in Fig. 28). Since the previous calculations of the field welds called for $^{5}/_{16}$-in. welds, it is clear that the end connection angles, having a length of 12 in., are satisfactory for the conditions assumed in this example.

The next type of connection which will be considered is the so-called seat angle. This is a familiar connection in riveted construction and is one of the simplest and most satisfactory methods of supporting a beam in welded construction whenever it can be used. Figure 54 (A) shows a typical connection of this type. The problem is to select a point of application for the beam reaction, W, so that the welds may be calculated according to the method given in Fig. 33. The web crippling formula of the American Institute of Steel Construction offers the most logical solution and is clearly set forth in Fig. 54 (D). The reaction is assumed to be uniformly distributed over the required length of bearing, N, hence the resultant of the reaction acts at a distance of $N/2$ from the end of the beam.

The action of a beam seat may be divided into the two steps of Fig. 54 (B). In the first step the beam deflects under the load and takes a slope at its support. This tends to throw all of the reaction on the toe of the seat angle. In the second step the outstanding leg of the angle has deflected until at some point it has attained the same slope as the end of the beam. The location of the point at which the load is then acting will depend upon the thickness of the angle. The thicker the angle, the farther out on the leg the load must act in order to bend it to the required slope. The next step in the problem becomes the determination of the thickness which will keep the load at or within the distance $N/2$ from the end of the beam, as found by the A.I.S.C. method.

Calculations readily show that the outstanding leg is stressed in bending to the yield point in being brought to the required slope. Let d = distance at which the load W produces yield-point stress. From the theory of plastic bending, at a distance of $3d/2$ distortion will be continuous without increase of load. In Fig. 54 (C) the distance $B = 3d/2$. Evidently then, if the thickness is made such that the yield stress (assumed as 33,000 psi.) is attained at a distance d, from the load, it will be impossible for the point of application of the load to exceed the distance, B. The section from which B is measured has been arbitrarily taken as $^{1}/_{8}$ in. from the inside face of the vertical leg. By equating the two distances from the back of the angle to the load, as shown in the

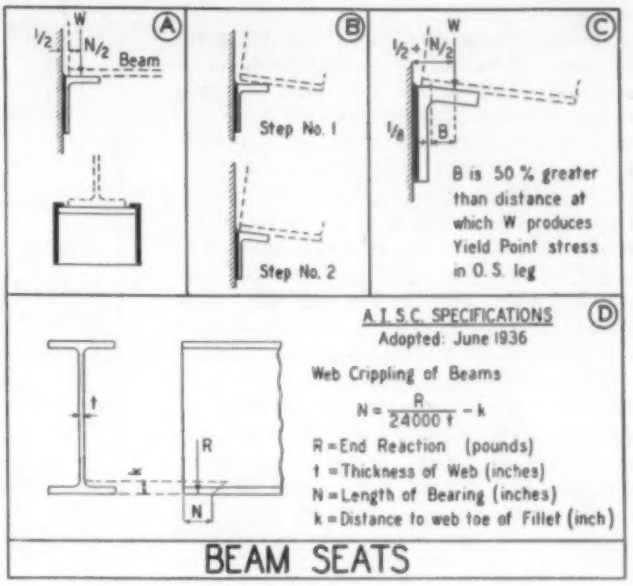

Fig. 54

figure, it becomes possible to arrive at a solution for the thickness.

The solution becomes a simple matter by the use of the chart in Fig. 55. For most beams under which a seat angle is practical, a length of angle between 8 and 10 in. is great enough to provide space for field welding the beam flanges to the seat. The chart has been prepared to cover lengths of angle, b, within these limits. Again it will be noticed that the welding has been carried over the top of the angle for a short distance. This is the point at which the bending stress on the welds is greatest and it is well to add this extra strength, although it has not been used in the calculations. The lengths of leg, L, given in the chart are the common width of angles. The example in the figure is sufficient to illustrate the use of the chart.

Assume a 14 WF 30 beam to have an end reaction of 18 kips. The flange width is 6.73 in.—thus b = 8 in. will be satisfactory length of seat angle. From the A.I.S.C. formula of Fig. 54 (D), using t = 0.270 in. and $K = {}^{7}/_{8}$ in., one-half the length of end bearing, N, is calculated to be 0.95, which added to the end clearance of $^{1}/_{2}$ in. makes the distance a equal to 1.45 in. Let it be further assumed that the welds are to $^{5}/_{16}$ in., having a working unit stress equal to 3.0 kips per inch. The coefficient, K, is found by dividing the working stress on the weld by the beam reaction. For the example $K = 3.0/18.0 = 0.167$.

The procedure for using the chart is indicated by the dotted line. First enter the lower left-hand side at W = 18.0 kips and move vertically to the intersection with the sloping line for b = 8 in., thence horizontally to the point at which a = 1.45 in. It will be seen that this point falls between the curves for $T = {}^{7}/_{16}$ and $^{1}/_{2}$ in. Using the larger value, the required thickness of the seat angle is $^{1}/_{2}$ in. Next follow upward vertically along the line a = 1.45 in. to the point K = 0.167. This point lies between the curves for L = 5 and 6 in., and we use the larger value. The seat angle is then fully determined and is an angle 6 x 4 x $^{1}/_{2}$ with a length of 8 in.

The use of unstiffened seat angles should probably be limited to a maximum load of 35 kips and for beams not exceeding 18 or 20 in. in depth. One important point should be emphasized, namely, that if any horizontal force occurs which tends to pull the seat angle away from its support, then additional welding must be added across the top of the angle to resist this force.

When the loads exceed the capacity of the "seat angle" connection it is general practice to use the stiffened seats or brackets shown in Figs. 56 (*A*) and (*B*). The first type uses a split beam section in which the web of the beam acts as the stiffener under the flange. The second type is built up from two pieces—a bearing plate and a bar stiffener. Calculations for the welding are given in Fig. 56 (*C*). The point of application of the load may be assumed as being at the distance $a = 0.8A$ from the face of the support. In order to have a proper amount of welding along the stiffener it is recommended that in the design calculations the distance d shall not exceed 20% of the length, L. When $d = 0.2L$ the moment of inertia of the two welds about the center of gravity is $I_0 = L^3/4$ and the section modulus is $S = 0.6L^2$. Figure 57 is a chart to facilitate the design of such brackets and its use is identical with that of the chart in Fig. 33.

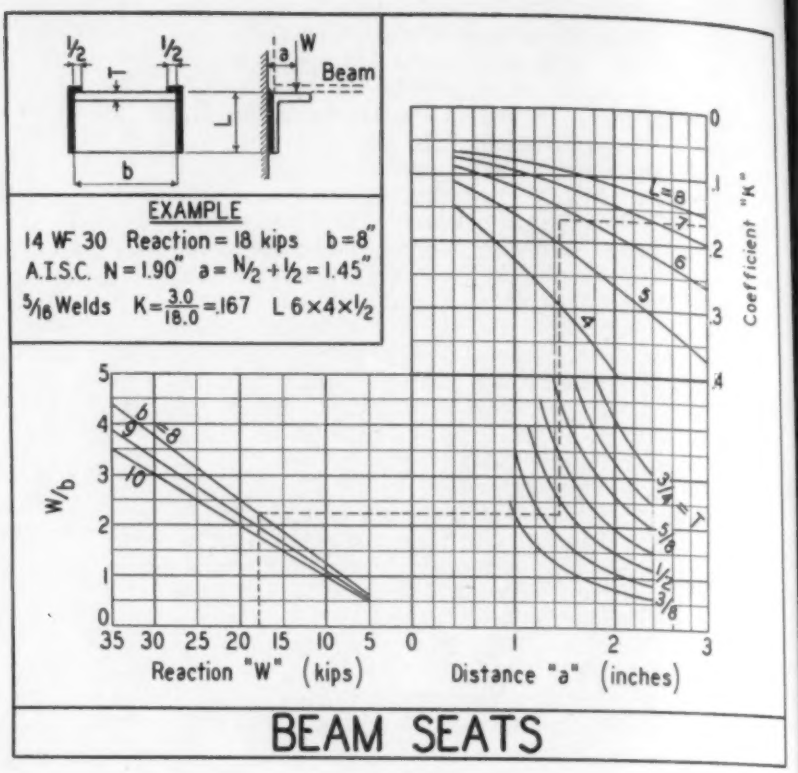

Fig. 55

In a paper entitled "Welded Structural Brackets"* by Prof. C. D. Jensen of Lehigh University the design method given above was confirmed on the basis of photoelastic studies and tests of steel prototypes. Prof. Jensen further proposed a method of obtaining the thickness of the stiffener by calculating the stress distribution on a section of the stiffener just below and parallel to the seat, based upon the familiar formula $f = P/A + M/S$. For the bracket shown in Figs. 56 and 57:

$$f = \frac{W}{At} + \frac{W \times 0.3A}{tA^2/6} = \frac{2.8W}{At}$$

If $f = 20$ kips per square inch in compression

$$t = \frac{0.14W}{A}$$

where

W = load in kips
A = width of stiffener in inches
t = thickness of stiffener in inches

End Rotation

In most structures, it is desirable and many times essential to connect the flanges of the beams so as to maintain alignment. The end rotation of beams shown in Fig. 49 then becomes of practical importance. The sketch in Fig. 58 (*A*) shows the end rotation for a uniformly loaded beam, which amounts to a displacement of 0.009 in. for each inch of distance from the center of rotation, assumed to be at the center line of the beam. For an 18-in. beam, the movement at the top, b, equals 0.081 in.

A small amount of welding placed on the top flange would be almost certain to crack as it would lack sufficient strength to resist the displacement from the rotation. When the fillet weld of sketch *a* is used the combination of elongation and the stress concentration at the root of the weld can easily be too much for the weld to stand. The stress path in the weld of sketch *b* is more direct, but here, too, the weld is beset with the same difficulties on a less intense scale.

End rotation is a very positive factor and there is no compromising with it. Either sufficient welding must be placed on the joint to entirely resist the rotation or some adequate provision made to permit a partial or complete yielding of the connection.

Several methods have been proposed and used for meeting this situation. One of the simplest and most common is the top angle connection of Fig. 59 (*C*). The vertical leg, against the column, is welded across the top edge and the weld carried around the ends for a short distance. Welding on the horizontal leg of the angle is

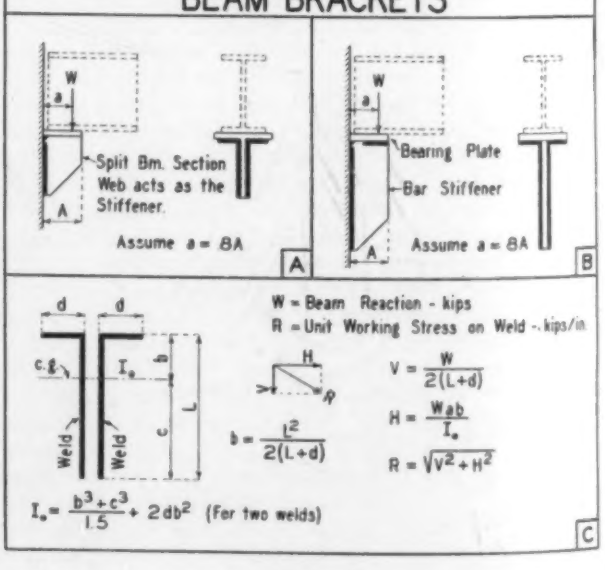

Fig. 56

* American Welding Society Journal, October 1936.

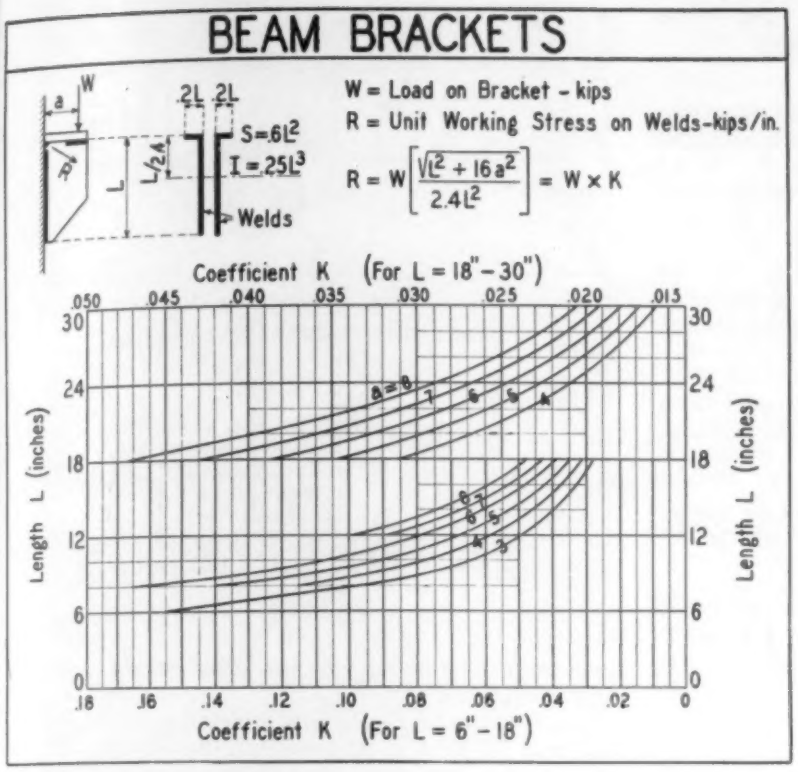
BEAM BRACKETS
W = Load on Bracket - kips
R = Unit Working Stress on Welds-kips/in.
R = W [√(L² + 16a²) / 2.4L²] = W × K
S=6L²
I=.25L³
Welds
Coefficient K (For L = 18"-30")
Length L (inches)
Coefficient K (For L = 6"-18")

Fig. 57

across the edge or toe and may be carried around the ends also. By keeping the thickness of the angle to a reasonable value, the welds are amply strong to produce the distortion shown in the top angle which takes up the movement of the flange and yet leaves the top flange supported by the connection.

Another method is shown in Figs. 59 (*A*) and 59 (*B*), the only difference between the two being in the length of the top plate. In both cases, most of the movement of the top flange is taken up in the elongation of the plate. It is certain that the stress in the plate will reach the yield point as the movement of the flange exceeds the elastic stretch of the plate for the lengths that are practical.

It must be clear that such connections as shown in Fig. 59 exert a horizontal pull on the top flange which, when coupled with a similar force at the connection of the bottom flange to the seat angle or bracket, produces a negative moment. Endeavors have been made to utilize this moment and its consequent reduction of the positive bending moment at the center of the span to achieve some weight saving in the required section of beam. An extensive series of tests on these and other connections is reported in an article "Tests of Miscellaneous Welded Building Connections" by Bruce Johnston and Gordon R. Deits, published in THE WELDING JOURNAL of January 1942. Anyone desirous of following the design procedure involving this moment-rotation relation should thoroughly study this article by Johnston and Deits. The authors state that the tests should not be considered as providing the final answer to the acceptability of any type of connection.

Continuous Structures

Continuous structures have been receiving considerable attention in recent years, as designers have sought to obtain economy of material, and at the same time a saving in total cost as compared to structures designed under the ordinary assumptions of simple supports. Continuity removes design from the realm of the hand book, for the engineer must have a sound working knowledge of the elastic behavior of structures, their distortions and the methods of analysis by which the stresses are determined. The calculations are more extensive and involved, and to a large extent must be based upon repeated trials, since the distribution of loading among the members of a continuous structure is dependent upon the relative stiffness of the members, which, in turn, is not definitely known until the sizes of the members have been determined.

This paper cannot encompass any complete or even comprehensive discourse on the subject of continuous structures. There are a number of textbooks that treat the subject at length. An excellent book *Analysis of Rigid Frames* by A. Amirikian, covers the application of the slope deflection method of analysis to a wide variety of rigid frames. The natural rigidity of welded joints has led designers to employ it in some welded structures, and there are articles in the files of the A.W.S. WELDING JOURNAL describing the construction, to which the reader is referred.

Let us first consider the problem of continuous beam spans. It is well known that when all spans of a series of spans of equal length, L, are loaded with a uniform load or W pounds per foot, and where the ends of the line of spans are fixed, the maximum bending moment equals $WL^2/12$. One frequently sees comparisons of weights of beams required when designed for this moment and for the moment of $WL^2/8$ for simply supported beams. When utilizing the rolled beam sections available from the mills, the saving in weight will average about 23%. Likewise, a moment of $WL^2/10$ will save about 13 or 14% in weight.

In practice, it is difficult to restrain a beam completely against end rotation. There are also factors which have an effect upon the distribution of the mo-

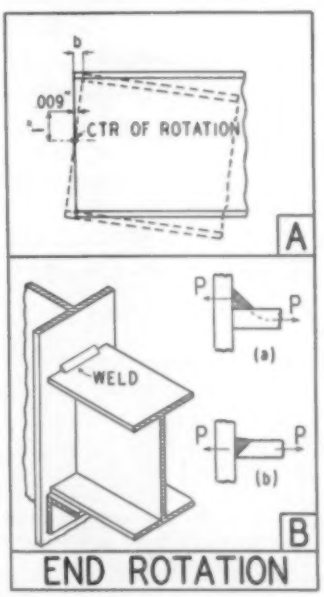

Fig. 58

ments that should be appreciated by the designer of continuous structures. Some of these are listed below.

1. Settlement of Supports:
   This may arise from the deflection of girder beams over which a line of beams is assumed to be continuous, as well as from the actual settlement of foundations or other physical supports.
2. Unequal Span Lengths:
   Such an arrangement results in an unequal distribution of bending moments and can even result in negative moment throughout a short span.
3. Partial Loading:
   In a series of spans, the placing of live loads on certain spans only results in unequal distribution of bending moments.
4. Partial Fixity:
   Where the ends of the beams cannot be wholly restrained, account must be taken of the resulting moment distribution.

These factors clearly emphasize the fact that no beam in any one span can be designed independently, using the moment $WL^2/12$. All beams and spans must be considered together, since there is an interaction among them that finally determines the proportion of the total moment that each will carry.

When the beams are supported by columns, either in single or multistory buildings, there is still another factor to be considered. In such structures, the columns share in the moment distribution and the calculations must include their effect. One result is usually an increase in the column section, over that obtained under usual methods of design, thus offsetting to a degree the saving in weight of the beams achieved by designing them for continuity.

When continuity has been decided upon as the method to be followed in designing a structure, the means of achieving it are, in essence, relatively simple. Provision must be made at the ends of the beams for transmitting the stresses resulting from the end bending moments. This requires the flanges to be connected with sufficient welding to carry the calculated stresses. Where the framing is of regular outline, the methods of calculations

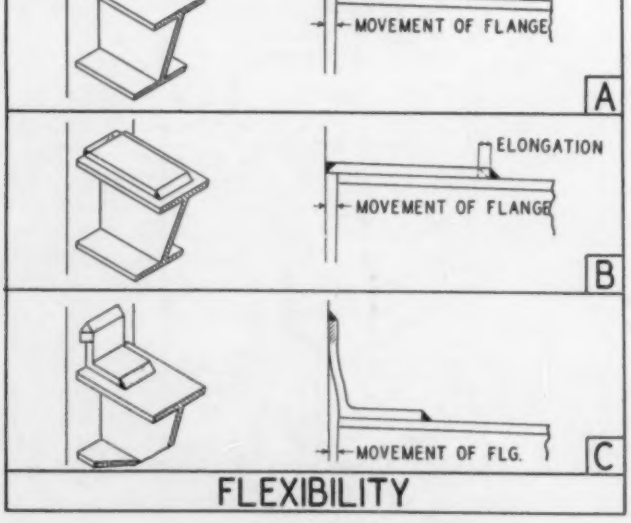

Fig. 59

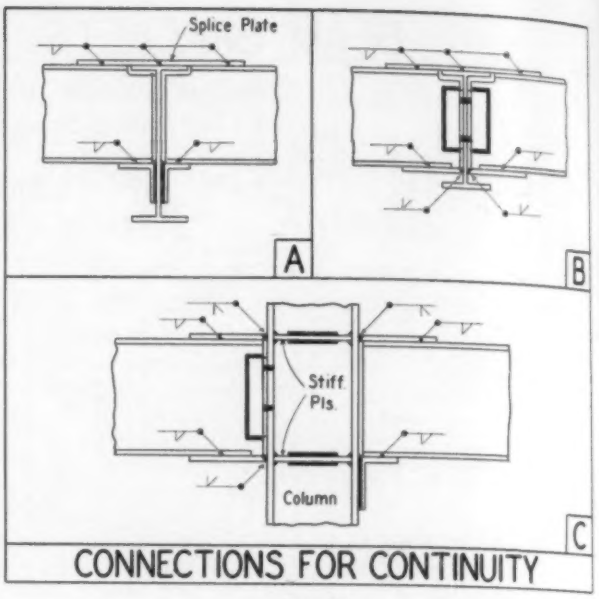

Fig. 60

have been considerably simplified by certain reasonable approximations.

The sketches of Fig. 60 are suggestive of connections that may be used in transmitting the stresses. Sketch *A* shows the condition when beams frame to a girder beam where there is sufficient room to use a seat angle to carry the vertical shear. The top flanges are tied together with a splice plate over the top of the girder. In many cases there may not be sufficient space available on the outstanding leg of the seat angle for all the welding that is required. This situation can be met either by fillers packed between the end of the beam and the girder web or by a gusset plate welded to the beam flange and bearing against the girder web. Of course it is assumed that the stress in the bottom flange is compression, but if there is any positive moment possible from reversal of loading, partial loading or the like; then provision must be made to transmit this tension stress by welding the top of the seat angle to the column or welding the inserted fillers to the beam and girder.

Sketch *B* illustrates a condition in which the vertical shear must be transmitted by a web connection such as shown in Fig. 51. A bottom gusset plate is shown for illustration. If one is used, proper clearance must be provided to make the weld between the gusset and the girder web.

When continuous beam connections must be provided at columns, as in sketch *C*, the details of the connections are quite similar to those of sketches *A* and *B*. Provision has to be made for transmitting the stresses over the space between the column flanges and this is accomplished by the insertion of stiffening plates as shown. Unless these plates are added, the outstanding flanges of the column will be bent by the connections on the beam flanges, resulting in loss of the continuity. When there are beams framing to the web of a column, the stiffener plates between the column flanges also serve to transmit the forces at the top and bottom flanges of these beams.

It has always seemed to the writer that continuity in beam framing is only achieved at the cost of a very considerable amount of welding, especially field welding which is expensive, and that its advantages should be carefully weighed against the total cost of the construction. Where total cost is secondary to weight saving or to definite reasons for desiring rigidity in the structure, the design may be based upon continuity from its inception.

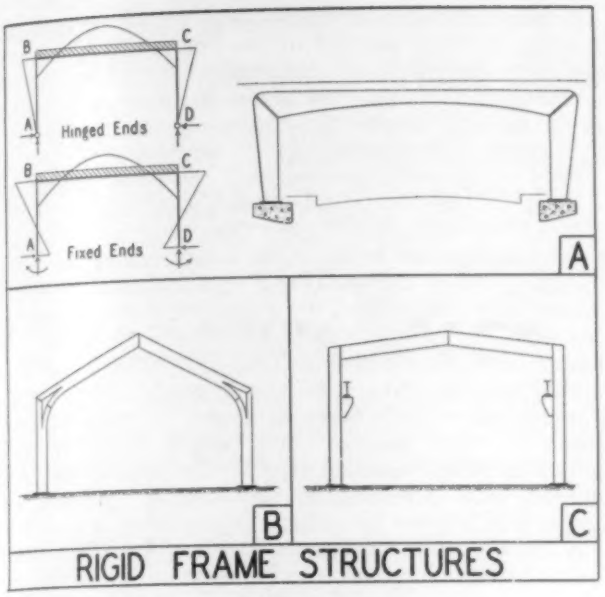

Fig. 61

Rigid Frames

There is a class of structures, known as rigid frames, which has found a place in modern structural design. Essentially, such structures are continuous and are well adapted to the utilization of the rigidity that is so readily obtainable with welded connections.

Figure 61 (*A*) illustrates a type of rigid frame bridge. This construction permits of a shallow depth at the center of the horizontal span which results in meeting the required headroom below the structure without increasing the height of abutments and approaches. In fact, the height may be reduced over that required for simple beam or girder spans. The general shape of these rigid frame bridges makes for pleasing architectural structures, many examples of which are to be seen in the parkways of Westchester County, New York.

The principles of design are shown in a very limited way by the moment diagrams of Fig. 61 (*A*). The supports at *A* and *D* may be hinged, or fixed, or partially fixed, depending upon the basic assumptions and the degree to which the foundation is designed to provide the fixation. The structure is continuous at the knee joints, *B* and *C*, and it will be noted that these joints are subject to bending moments, the amount depending upon the dimensions and section, which influence the distribution of the moments produced by the loads. The negative moments at the knees reduce the moment at the center of the span *BC*, thus reducing the section required and so providing one of the advantages previously mentioned.

Rigid frames in building construction are particularly desirable where large unobstructed floor areas are required and where lighting or visibility are important factors. Such buildings would include auditoriums, gymnasiums, armories, hangars and many types of industrial buildings. A building bent, such as shown in Fig. 61 (*B*), provides clear floor space without the use of trusses that often limit vision in auditoriums or require greater height of building to gain the desired range of vision.

The building of Fig. 61 (*C*) is a very elementary cross section, which has been the basis for expansion into buildings with multiple aisles and with intermediate floors. The absence of knee braces requires less height for crane clearance, and when one considers the cost of extra height of walls required with conventional construction, the advantages of a rigid frame construction may be considerable.

In all such structures it becomes difficult to draw a line between a beam and a column. All members of a rigid frame carry moments and all may be subject to direct tension or compression. Hence the design of all members involves the selection of a section that will be subject to combined stresses.

It is clear that the lower ends of the vertical supporting members (columns) will have a tendency to spread outward under the action of vertical loads on the span. The horizontal force required to restrain this motion is often supplied by a tie rod between the columns and below the ground or floor level.

No attempt will be made in this paper to discuss the details of the design of rigid frames. Several references to articles and books, listed in the bibliography, will provide the interested reader with considerable information on this subject.

Fatigue

In structures subject to vibration, repeated or cyclic stresses, or dynamic shocks or impact, there are failures which often occur at calculated stresses below those used in the design. Little or no perceptible elongation is exhibited by the broken parts and the fractured surfaces have a characteristic appearance, generally being made up of two rather distinct portions. One appears smooth with slight ripples that seem to be more or less concentric about a nucleus or focal point, usually on the original surface of the member. The second part of the fracture appears crystalline. The first represents the slow, progressive failure, while the second represents the sudden, final failure when the section had been so reduced that it no longer could carry the load. This type of failure is known as "fatigue" failure.

Designers of welded structures must have a good knowledge of the engineering principles that underlie correct design to avoid fatigue failure. A study of any fatigue failure will almost invariably lead to the primary cause being a notch, nick, scratch, tool mark, steel stamp impressions, sudden change of section or any discontinuity in the material. All of these constitute stress raisers which are the creators of stress concentrations. In the section of this paper dealing with "Stress Paths" it was pointed out that welds (particularly fillet welds) are potential stress raisers—the severity of the resulting stress concentrations depending upon the lack of smoothness of the surface, lack of penetration, presence of internal defects or blowholes, excessive reinforcement and undercutting of edges.

It was shown that the intensity of stress concentration may be several times the average stress and when these high stresses at the root of notches, etc., are repeatedly induced by the loading, we have the formation of a minute local crack which is subsequently propagated as the metal at the base of the crack is cold worked and weakened by the successive applications of the stresses.

The subject of the fatigue strength of welded joints has received increasing attention and Bulletins 310 and 327 of the University of Illinois Engineering Experiment Station give the results of tests on large butt-welded specimens. This work to date is summarized in Report No. 3 of the Committee on Fatigue Testing (Structural) of the Welding Research Council of the Engineering Foundation, published in THE WELDING JOURNAL for May 1943. These publications are well worth study by engineers.

The conclusions stated in Report No. 3 include the following quotations:

"(1) Butt welds in $^7/_8$-in. carbon steel plate, of the quality of A.S.T.M. Specification A7, welded in shop under fairly ideal conditions of flat position, skilled operators, smooth contours, single-U grooves, back-chipped, back-welded and revealing virtually no porosity under X-ray, developed, in the as-welded condition, fatigue strengths equal to or greater than:

Cycle zero-to-tension	* N = 100,000	32,000 psi.
	N = 2,000,000	22,100 psi.
Complete reversal	N = 100,000	21,400 psi.
	N = 2,000,000	13,300 psi.

* N = number of cycles.

(2) For butt welds in $^7/_8$-in. carbon steel plate of the quality of A.S.T.M. Specification A7, with carbon not over 0.25% nor manganese over 0.70, commercially butt welded in accordance with the A.W.S. Specifications for Welded Highway and Railway Bridges, and in compliance with all the specification requirements for inspection of operators, and for workmanship, whether welded in shop or field, and in any position, there may be anticipated, in the as-welded condition, "dependable" fatigue strengths as follows:

Cycle zero-to-tension	N = 100,000	27,000 psi.
	N = 2,000,000	18,000 psi.
Complete reversal	N = 100,000	16,500 psi.
	N = 2,000,000	11,000 psi.

(4) For butt welds as described under either (1) or (2) above, it may reasonably be expected, from the few tests made in the "basic" series, that the fatigue strength for the cycle, max. tension to $^1/_2$ max. tension, for 2,000,000 cycles, will exceed the static yield point at the base material.

(5) For butt welds showing surface defects such as undercutting and marked abruptness of reinforcement contour, and for butt welds lacking in completeness of fusion, the fatigue strength may be expected to be less than stated in (2) above and cannot be dependably predicted. For butt welds not complying in procedure and workmanship with the specification stated in (2) above, and not equal to the better or best commercial welds in fusion and homogeneity, no dependable prediction can be made as to the endurance under repeated loading. No such butt weld should be permitted in a bridge or other intermittently loaded structure."

Conclusion 5 is eloquent testimony to the importance of stress raisers in welded construction. It must be evident that the remedy for most fatigue failures lies in taking precautions to avoid stress concentrations. Many of them are self-evident. A few will be briefly discussed for the light they may shed on the problem. It is also well to re-examine Figs. 22, 23 and 24 in view of the preceding discussion.

The tests on butt welds in steel plates, conducted at the University of Illinois showed an increased fatigue strength for the joint when the reinforcement was ground off flush with the plate. This finding points to the advantage to be gained by removing a stress raiser by grinding, or even machining, when the welded member is subject to fatigue conditions in service.

When end fillets (as in Fig. 22 (*C*)) lie across the line of stress, it has been found in tests that grinding the fillet to a concave contour results in an increase of fatigue strength. The reason is clear when one considers the relative conditions in Figs. 22 (*A*) and 22 (*B*)—the latter being relieved of the high stress concentration to be found at point *a* of Fig. 22 (*A*).

In these grinding operations it is important that the grinding be done parallel to the line of stress. Otherwise, the inevitable grinding marks constitute minute notches and their position across the path of the stress would simply provide the stress raisers that are the forerunners of fatigue failures.

Another "stress-concentration" condition was shown to exist in longitudinal fillet welds (Fig. 18) where the ends are stressed much higher than the average for the entire length of the weld. This is inherent in the elastic deformations of the weld and base metal. The intermittent welding of web to flange in plate girders, as shown in Fig. 39, is satisfactory for static loads, but fatigue tests of such girders have shown that continuous welding provided greater fatigue strength. With a series of intermittent welds there are a series of twice as many points of stress concentration, which, combined with the stress raisers associated with the form of the weld contour and penetration, undoubtedly provide the factors which lower the fatigue strength.

It has been previously mentioned under "Butt-Strap Splices" that pointed straps, as shown in Fig. 36 (*E*), are lower in fatigue strength than such a splice as in Fig. 36 (*C*). The same would apply to the ends of cover-plates on beams or girders.

At times it may be possible to locate welds away from points of high stress or in regions where the distortions or deflections are small. Such steps should always be given consideration.

The subject of "Fatigue" is not one to be covered in such a limited space as this paper affords, but possibly enough has been mentioned to impress the reader with its importance in connection with any structure in which fatigue conditions exist. Engineering literature contains a wealth of material on the topic. In addition to the references to the bulletins of the University of Illinois Engineering Experiment Station, the writer would call particular attention to the book *Prevention of Fatigue of Metals* by the Staff of the Battelle Memorial Institute. The discussion of "Unit Stresses for Welded Design" in Appendix A of the A.W.S. Specifications for Welded Highway and Railway Bridges will be found most helpful in the study of this subject.

Conclusion

As was true of the first paper, there are many topics which have not been treated. Some have been discussed rather briefly, but the writer hopes that the reader will gain a good working knowledge of some of the fundamentals that underlie sound welding design. While structures have been the main background, it is well to emphasize that the general principles are applicable over wide fields of industrial construction.

Acknowledgments

The writer wishes to acknowledge and express his appreciation to the following for permission to use certain illustrations in the text of the paper:

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Figure 4. The Lincoln Electric Co.
Figures 20 and 21. Dr. Max M. Frocht, Carnegie Institute of Technology.

Numerous references to publications of the AMERICAN WELDING SOCIETY are to be found throughout the text. Permission to quote freely from this source was generously granted.

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Cause and Cures of 14 Common Welding Troubles

By C. H. Jennings*

THERE are many fabricating difficulties encountered in the arc welding of any structure. Some of these difficulties seriously affect the strength and serviceability of the ultimate structure while others are less important and only influence the cost or appearance.

The correction of these troubles is generally not difficult providing the welding operator or engineer has a knowledge of the conditions causing them. In order to assist in detecting and correcting these undesirable factors, 14 of the more common troubles are illustrated by photographs and discussed from the standpoint of causes and methods of correction.

CAUSE	*CURE*
Trouble—Distortion—Fig. 1	
A. Shrinkage of deposited metal pulls parts together and changes relative positions.	A. Properly clamp or tack parts to resist shrinkage.
B. Non-uniform heating of parts during welding causes them to distort before welding is finished. Final welding of parts in distorted position prevents the maintenance of proper dimensions.	B. Pre-form parts sufficient to compensate for shrinkage of welds.
C. Improper welding sequence.	C. Distribute welding to prevent excessive local heating. Preheating desirable on some heavy structures.
	D. Removal of rolling or forming strains before welding is sometimes helpful.
	E. Study structure and develop a definite sequence of welding.
Trouble—Warping (Thin Plates)—Fig. 2	
A. Shrinkage of deposited weld metal.	A. Select electrode with high welding speed and moderate penetrating properties.
B. Excessive local heating at the joint.	B. Weld rapidly to prevent excessive local heating of the plates adjacent to the weld.
C. Improper preparation of joint.	C. Do not have excessive spaces between the parts to be welded.
D. Improper clamping of parts.	D. Properly clamp parts adjacent to the joint. Use back up to cool parts rapidly.
	E. Use special welding sequence; step back or skip procedure.

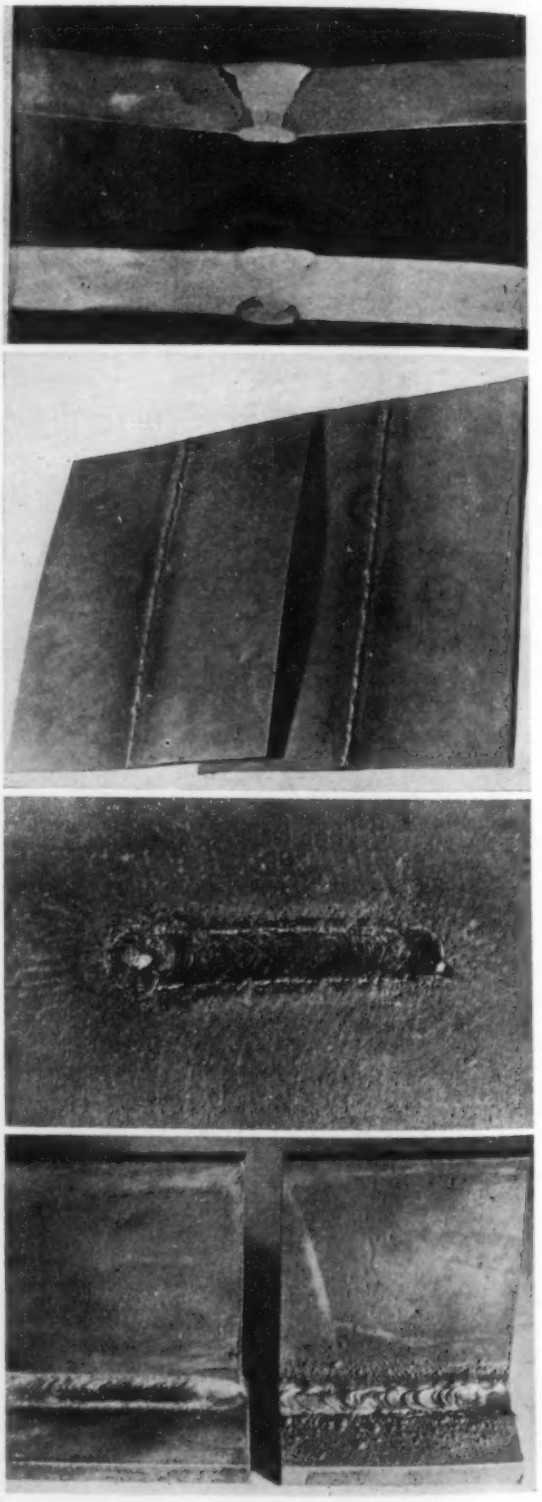

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Fig. 1—(Top) Angular Distortion Obtained with Single Vee Butt Joint Not Properly Prepared or Clamped. (Bottom) Single Vee Butt Joint Free from Angular Distortion, Accomplished by Proper Clamping

Fig. 2—(Overlapping) Warped Thin Plate, Improper Tacking and Welding Procedure. (Underneath) This Plate Free from Warping, Properly Tacked and Clamped, Welded by Skip and Step-Back Method

Fig. 3—Lines Radiating from the Weld Are Located Where the Mill Scale Has Cracked Off of the Plate Because the Residual Stresses Produced by the Weld Shrinkage Have Exceeded the Yield Point of the Material

Fig. 4—(Left) Weld Made with Proper Current and Arc Length; Note Absence of Adhering Splatter. (Right) Weld Made with Excessive Current and Arc Lengths; Note Excessive Splatter and Poor Appearance

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CAUSE	CURE
	F. Peen joint edges slightly before welding. This elongates edges and the weld shrinkage causes them to pull back to the original shape.
Trouble—Welding Stresses—Fig. 3	
A. Joints too rigid. B. Improper welding sequence. C. Inherent in all welds, especially in heavy parts.	A. Slight movement of parts during welding will reduce welding stresses. B. Make weld in as few passes as practical. C. Peen each deposit of weld metal. D. Anneal finished product at 1100–1200° F. for one hour per inch of thickness. E. Develop welding procedure that permits all parts to be free to move as long as possible.
Trouble—Spatter—Fig. 4	
A. Inherent property of some electrodes. B. Excessive welding current for the type or diameter of electrode used. C. Excessively long arc. D. Arc blow.	A. Select proper type of electrode. B. Do not use excessive welding current. C. Hold proper arc length. D. Reduce arc blow. E. Paint parts adjacent to weld with white wash. This prevents spalls from welding to parts and makes removal easy.
Trouble—Undercut—Fig. 5	
A. Excessive welding current. B. Improper manipulation of electrode. C. Attempting to weld in a position for which the electrode is not designed.	A. Use a moderate welding current and do not try to travel too rapidly. B. Do not use too large an electrode. If the puddle of molten metal becomes too large, undercut may result. C. Excessive weaving will cause undercut, consequently it should not be used. D. A uniform weave will aid greatly in preventing undercut in butt welds. E. If an electrode is held too near the vertical plane when making a horizontal fillet weld, undercut may be obtained on the vertical plate.
Trouble—Crack Welds—Fig. 6	
A. Joint too rigid. B. Welds too small for size of parts joined. C. Poor welds. D. Improper preparation of joints. E. Improper electrode.	A. Design the structure and develop a welding procedure to eliminate rigid joints. B. Do not use too small a weld between heavy plates. Increase the size of welds. C. Do not make welds in string beads. Make weld full size in short section 8 to 10 in. long. D. Welding sequence should be such as to leave ends free to move as long as possible. E. Insure that welds are sound and the fusion is good. F. Preheating parts to be welded sometimes helpful. G. Prepare joints with a uniform and proper free space. In some cases a free space is essential. In other cases a shrink or press fit may be required.

Fig. 5—(Right) Undercut on Fillet Weld. (Left) Good Fillet Weld, Complete Penetration, Flat Face and No Undercut

Fig. 6—Cracked Weld Caused by Rigid Joint

Fig. 7—Weld with Poor Surface Appearance Caused by Excessive Welding Current and Improper Weave

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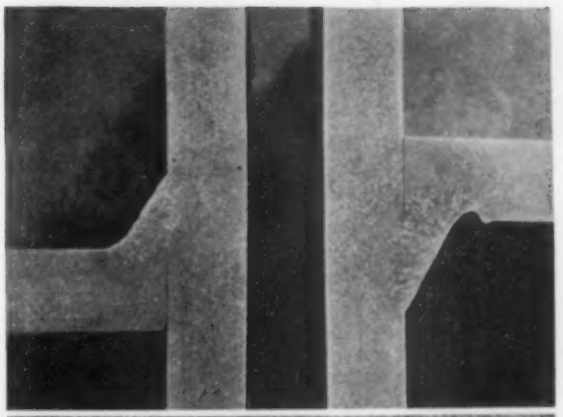

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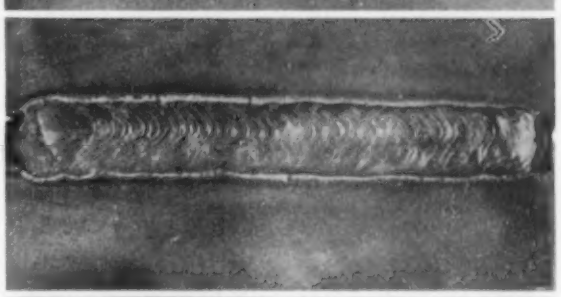

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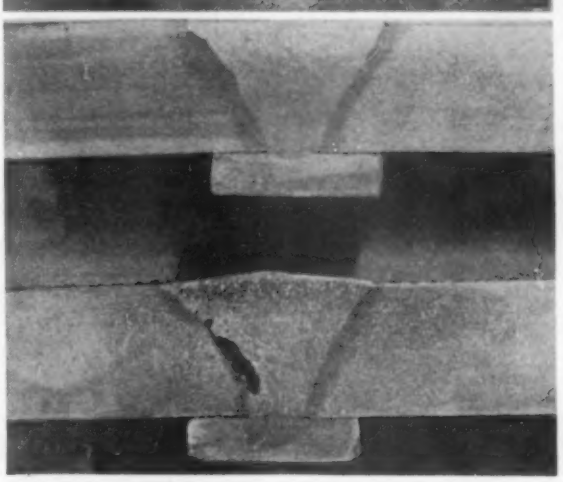

Fig. 8—Weld with Good Surface Appearance

Fig. 9—(Top) Good Butt Weld, Complete Fusion, and Penetration. (Bottom) Butt Weld with Poor Fusing and Slag Inclusions on Side, Improper Weaving Procedure and Low Welding Current

CAUSE	*CURE*
Trouble—Poor Surface Appearance—Figs. 7 and 8	
A. Improper current and arc voltage.	A. Insure the use of the proper welding technique for the electrode used.
B. Overheated work.	B. Do not use excessive welding currents.
C. Poor electrode manipulation.	C. Use a uniform weave or rate of travel at all times.
D. Inherent characteristic of electrode used.	D. Prevent overheating of work.
Trouble—Poor Fusion—Fig. 9	
A. Improper diameter of electrode.	A. When welding in narrow vees use an electrode small enough to reach the bottom.
B. Improper welding current.	B. Use sufficient welding current to deposit the metal and penetrate into the plates. Heavier plates require higher current for a given electrode than light plates.
C. Improper preparation of joint.	C. Be sure the weave is wide enough to melt thoroughly the sides of a joint.
D. Improper welding speed.	D. The deposited metal should tend to sweat onto the plates and not curl away from it.
Trouble—Incomplete Penetration—Fig. 10	
A. Improper preparation of joint.	A. Be sure to allow the proper free space at the bottom of a weld.
B. Use of too large an electrode.	B. Do not expect excessive penetration from an electrode.
C. Insufficient welding current.	C. Use small diameter electrodes in a narrow welding groove.
D. Too fast a welding speed.	D. Use sufficient welding current to obtain proper penetration. Do not weld too rapidly.
Trouble—Porous Welds—Fig. 11	
A. Inherent property of some electrodes.	A. Some electrodes inherently produce sounder welds than others. Be sure the proper electrodes are used.
B. Not sufficient puddling time to allow intrapped gas to escape.	B. Puddling keeps the weld metal molten longer and often insures sounder welds.
C. Poor base metal.	C. A weld made of a series of string beads is apt to contain minute pinholes. Weaving will often eliminate this trouble.
D. Too short an arc length.	D. Do not use excessive welding currents.
	E. In some cases the base metal may be at fault. Check this for segregations and impurities.
	F. Do not hold too short an arc.
Trouble—Brittle Joints—Fig. 12	
A. Air hardening base metal.	A. When welding on medium carbon steel or certain alloy steels the heat-affected zone may become hard as a result of rapid cooling. Preheating at 300–500° F. should be resorted to before welding.
B. Improper preheating.	B. Multiple layer welds will tend to anneal hard zones.
C. Unsatisfactory electrode.	C. Annealing at 1100–1200° F. after welding will generally soften hard areas formed during welding.
	D. The use of austenitic electrodes is sometimes desirable on steels which harden readily. The increase weld ductility compensates for the brittle heat-affected area in the base metal.

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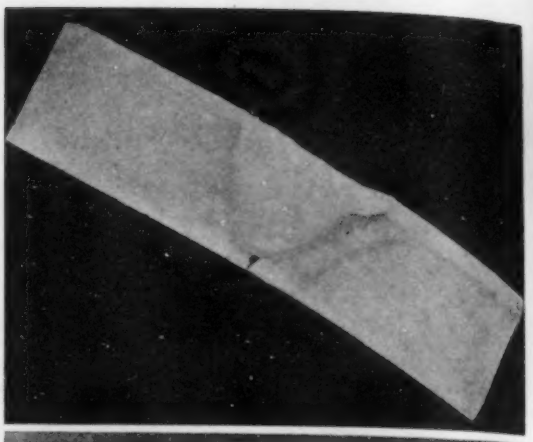

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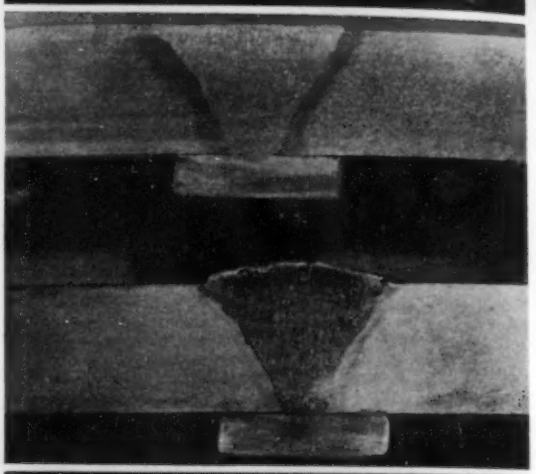

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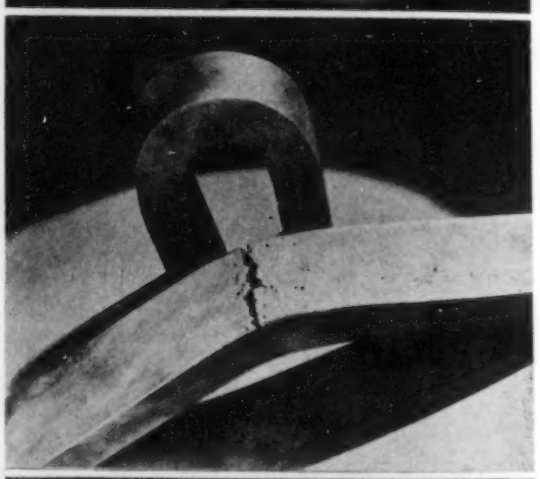

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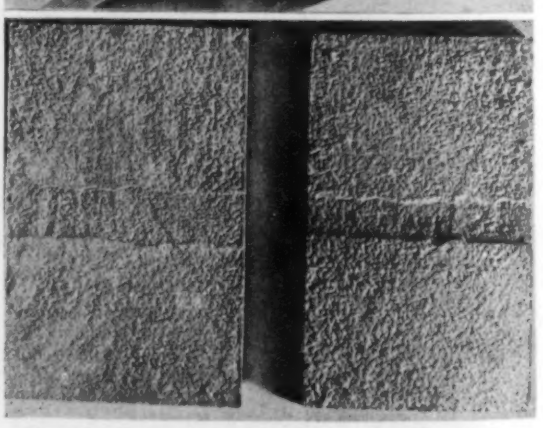

Fig. 10—Incomplete Penetration on Vee Butt Joint
Fig. 11—(Top) Sound Butt Weld. (Bottom) Porous Butt Weld Caused by Short Arc and Insufficient Puddling
Fig. 12—Ductile and Brittle Joints
Fig. 13—(Left) Corrosion of Welded Joint (HNO_3); Note Weld Is Almost Comparable to Parent Metal. (Right) Improper Electrode Causes Poor Corrosion Resistance of Weld (HNO_3)

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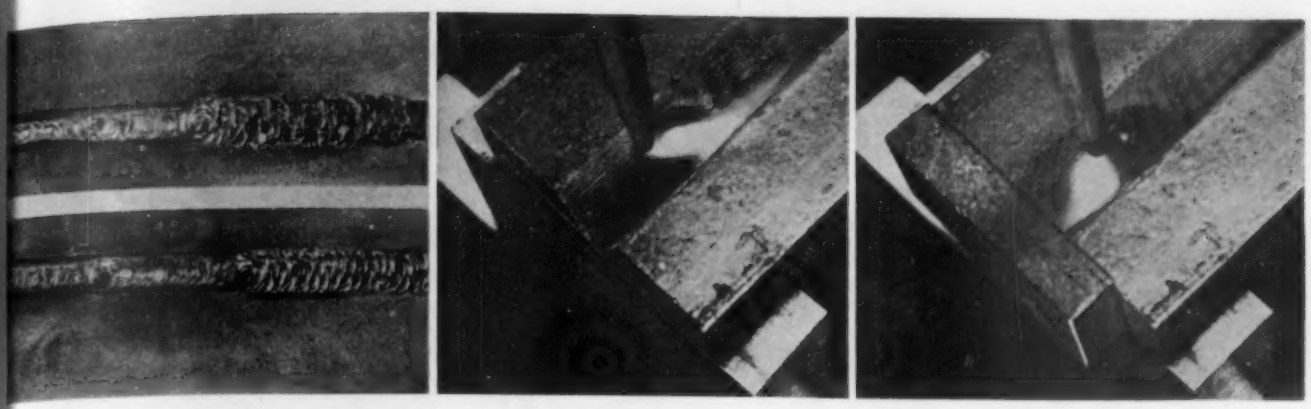

Fig. 14-(Top) Poor Vertical Weld-Improper Electrodes, Welding Technique.
(Bottom) Good Vertical Weld-Proper Electrode, Current and Electrode Manipulation.
Fig. 15—Arc Blow at End of Joint
Fig. 16—Arc Blow Controlled by Use of a Magnetic Shunt (Block of Steel at End of Joint)

CAUSE	*CURE*
Trouble—Corrosion—Fig. 13	
A. Type of electrode used.	A. Bare type electrodes produce welds that are less resistant to corrosion than the parent metal.
B. Improper weld deposit for corrosive media.	B. Shielded arc type electrodes produce welds that are more resistant to corrosion than the parent metal.
C. Metallurgical effect of welding.	C. Do not expect more from the weld than you do from the parent metal. On stainless steels use electrodes that are equal or better than the base metal.
D. Improper cleaning of weld.	D. When welding 18-8 austenitic stainless steel be sure the analysis of the steel and welding procedure are correct so that welding does not cause carbide precipitations. This condition can be corrected by annealing at 1900-2100° F.
	E. Certain materials such as aluminum require careful cleaning of all slag to prevent corrosion.

CAUSE	*CURE*
Trouble—Irregular Weld Quality—Fig. 14	
A. Improper electrode manipulation.	A. Use a uniform weave or rate of travel at all times.
B. Excessive welding current.	B. Do not use excessive welding currents.
C. Welding in improper position for which electrode is designed.	C. Use an electrode designed for the type of weld and the position in which the weld is to be made.
D. Improper joint design.	D. Prepare all joints properly.
Trouble—Magnetic Arc Blow—Figs. 15 and 16	
A. Magnetic fields cause the arc to blow away from the point at which it is directed.	A. Proper location of the ground on the work. Placing the ground in the direction the arc blows from the point of welding is often helpful.
B. Magnetic blow is particularly noticeable with d. c. at ends of joints and in corners.	B. Separating the ground in two or more parts is helpful.
	C. Weld toward the direction the arc blows.
	D. Hold a short arc.
	E. Change magnetic path around arc by using steel blocks.
	F. Use a.-c. welding.

FUNDAMENTALS OF WELDING METALLURGY

A brief textbook on the A, B, C in Metallurgy which enables one to understand articles on Metallurgy and to appreciate the Metallurgical problems involved in welding. The book—357 pages—is based upon a series of twenty lectures given at Polytechnic Institute of Brooklyn in cooperation with the New York Section of the American Welding Society. The book is bound practically in imitation blue leather covers. Price of the book is $1.50 per copy. These lectures were prepared by Professor Otto H. Henry of the Polytechnic Institute of Brooklyn and Dr. G. E. Claussen of the same University, who was also connected with the Welding Research Council.

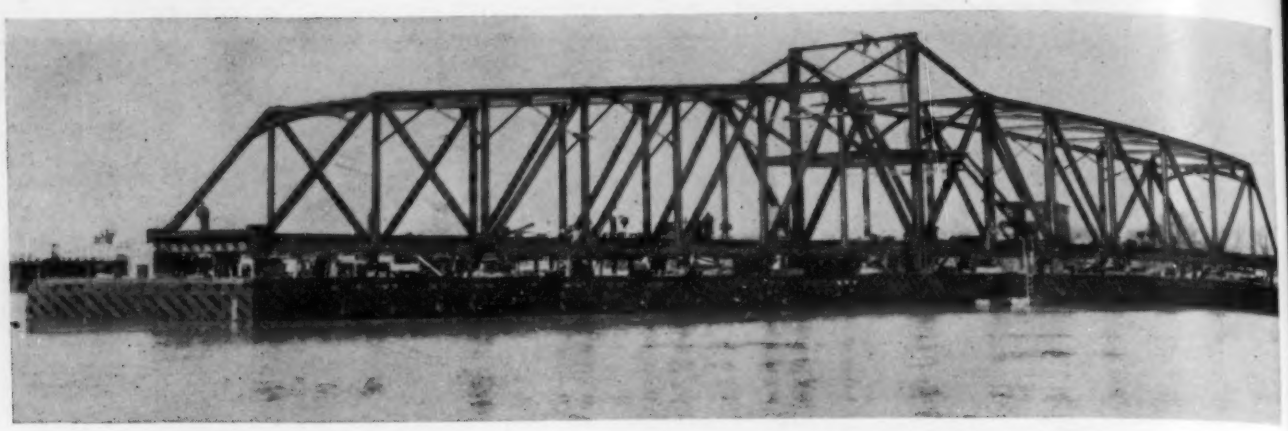

Fig. 1—Swing Bridge on Highway 12, North of Terminous, Across the Mokelumne River Near to Completion. The Structure Is Protected from Crashes by a Timber Fender, with 110-Ft. Navigation Channels on Each Side of the Bridge

Swing Bridge Is Semi-Welded*

By B. M. Shimkin†

ARC welding was used to a limited extent on a swing bridge built by the California Division of Highways in 1942 across the Mokelumne River about 5 miles northwest of Terminous on State Highway 12. In the course of design investigations for the structure, numerous matters of interest developed as riveted, welded, and semi-welded types of through-truss highway bridges were studied. Some of these matters are presented in this article.

The semi-welded design was finally adopted for this structure. It was designed in the office of the California Division of Highways in Sacramento, and erected in 1941–42. It was opened to traffic on June 1, 1942.

A duplicate swing bridge is expected to be built during this year at another site. A similar span, all-riveted swing bridge was built in 1936 on Highway 15.

In the construction of the present swing bridge arc welding was permissible for use within limits prescribed by the American Association of State Highway Officials, Specifications of 1938.

The total amount of arc welding used in the structure amounted to 2032 ft. (mostly $^5/_{16}$ in.) or 438 lb. of electrodes.

The soil conditions at the site are such that the height of the road fill is limited and total length of structure was fixed, as well as the grade of the bridge deck. This bridge crosses a navigable river and must be open for passing ships.

Different Types Studied

With such limitation an important consideration, comparative estimates of swing and bascule bridges were prepared and showed a saving in favor of swing bridges amounting to 4.8%.

The swing bridge described here was built in a lowland country, where the soil has very poor bearing capacity and an unlimited space for this kind of bridge was provided. The roadway width on the structure is 26 ft. between curbs and sidewalks 2 ft. wide were built on either side. Modified Pratt trusses were used, 31 ft. 9 in. between centers of trusses, and 287 ft. $3^1/_2$ in. between abutment bearings. The center bearing was based on a center round reinforced concrete hollow pier 40 ft. $6^7/_8$ in. in diameter set on 143 Douglas fir piles.

The center pier and abutments are protected by timber fenders. For navigation, two 110-ft. clearances are provided.

Design Studies

All machinery, abutments, piers and other details which are common for both welded and semi-welded designs are omitted from consideration here. However, it should be mentioned that the machinery weights expressed in percentages of structural steel weights for highway bridges equal to from 12 to 13% for span of 200 to 300 ft. This was checked in the design of the present bridge to give 11.7%, and is also given in Table 3L of "Movable Bridges" by O. E. Hovey. The figure of 12% was used in the final estimate for a welded structure.

For better comparison of the two designs, the controlling dimensions and general features of the welded design were made the same as for the semi-welded one. A single exception was made for the deck slab of welded design by using "I-beamlok armored bridge roadway slab," filled with Haydite concrete, instead of reinforced concrete slab such as was used in the present bridge. It should be mentioned also that the design of a swing bridge is quite different from that of a common truss bridge. It is not only a bridge, but a bridge and a great deal of machinery and special equipment (locks, buffers, gates, signals, operator's house, etc.).

In order to obtain the design stresses for truss members in a swing span it is required to calculate stresses in five loading cases and later five combinations of such cases. After that many members should be checked for additional wind, bending and eccentric loading stresses. These additional stresses considerably increase section areas of members compared with areas required by the "design stresses" only.

* Data and illustrations are from a study submitted by the author to The James F. Lincoln Arc Welding Foundation in its recent Industrial Progress Award Program for reports of advancements and improvements made by the application of arc welding in design, fabrication, construction and maintenance.

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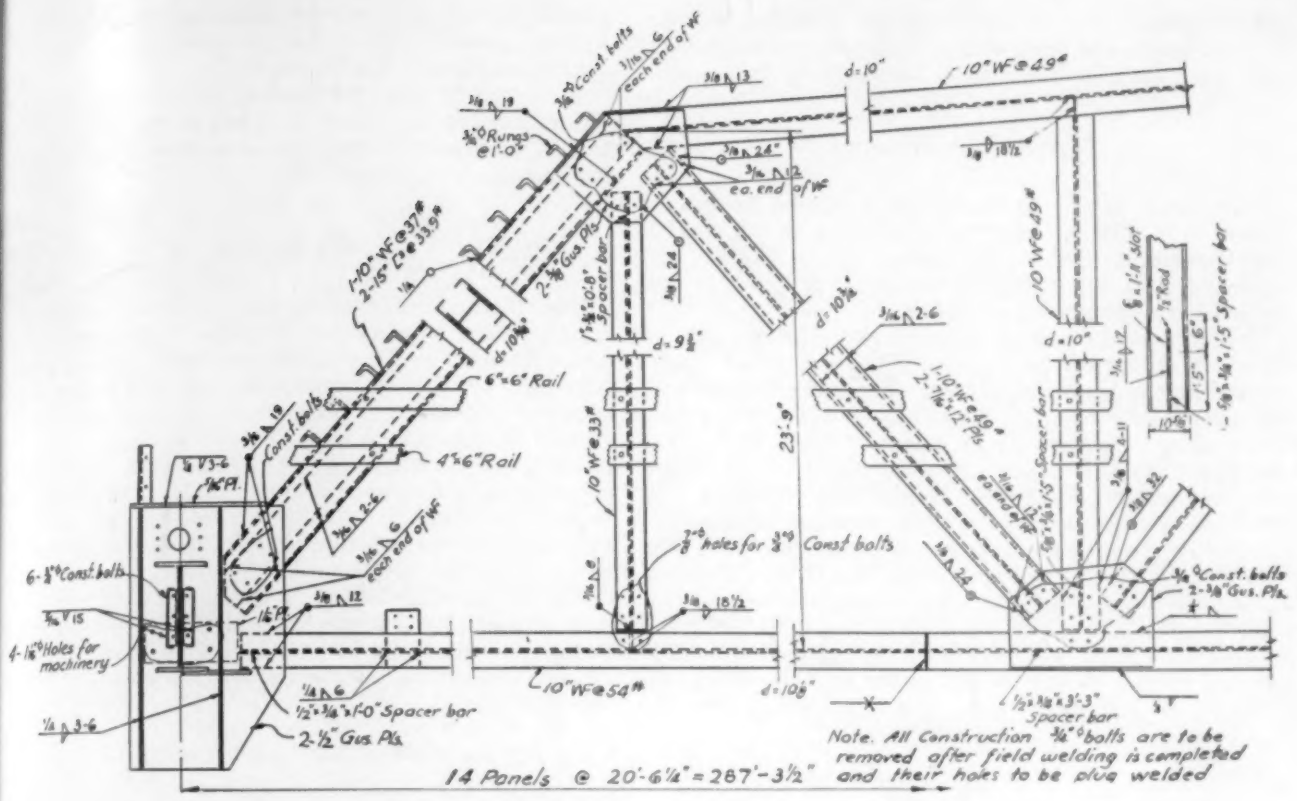

Fig. 2—A Part Elevation of an All-Welded Truss

Table 1—Comparison of Weight of Riveted and Welded Design
(Weight in Pounds)

Type of Truss	Top Chord	Bottom Chord	Verticals	Diagonals	Gusset Pls. Fillers and Splices	Rivet Heads* or Welds	Top Bracings	Bottom Bracings	Portal and Sway Bracings	Floor System†	Pivot and Ring Girders	Control House and Misc.	Total
Riveted	75,632	75,496	61,417	63,724	46,573	4765	42,508	20,839	28,779	222,003	85,423	31,851	759,000
Welded	48,632	47,520	50,550	55,000	21,940	1398	25,320	14,928	20,955	203,904	32,154	31,851	569,782
Difference in favor of weld	27,000	27,976	10,865	8,724	24,633	3367	17,188	5,911	7,824	18,099	53,269	....	189,218
Per cent	35.7	37	17.7	13.7	53	70	40.5	28.3	27.2	8.2	62.4	....	25.5

* In riveted truss welding was used in "built-up" members and in connecting the filler plates to gusset plates or members.
† Floor system included the girders, stringers, details.

Construction Methods

The presence of the rest draw and fenders in a swing bridge create the most ideal conditions for an erection of trusses. The contractor has a platform to work on without any extra expense for false work, in fact, he is paid for its erection.

The welded members and fillets to gusset plates in this bridge were shop fabricated and no field welds were used in its construction. However, electricity was available on the site from the beginning of work.

The absence of rivet heads on welded members gives a smooth appearance, creates a beautiful impression when compared with riveted members and decreases painting expenses; it also increases the life of the bridge by elimination of many "starting points" for rust around rivet heads.

In the case of full welded trusses the benefit of the rest draw will be extended by omitting the seat angles for the floor girders, stringers and other members, and supporting of all these members directly from the unyielded platform for the field welds, thus saving a great deal of construction expense. Again, the rest draw gives to the welders and their equipments the best working conditions compared with any other construction on false work and will increase their efficiency.

No filler plates are used in new welded trusses. They are replaced by the spacer bars, which give simpler details and eliminate the ambiguous design necessary for filler plates.

The AMERICAN WELDING SOCIETY formulas for calculation of the required area of welds for end connections of the truss members give quite large areas for welds and such excess creates a total amount of field welds greater than shop welds.

In the present case this disadvantage of field welds would not have increased considerably the cost of construction, because (1) electricity was available on the site; (2) presence of the rest draw gives the best conditions for work; (3) simplicity of erection; and (4) local perfect climatic conditions—and all these factors tending to insure about the same surroundings as in the shop.

General procedure of construction for welded trusses is practically the same as for riveted ones—start from the center pivot girder and proceed to the ends—the work to be executed according to the best present welding practice and the specifications of the AMERICAN WELDING SOCIETY and the American Association of State Highway Officials.

Observation of construction of the semi-welded trusses on this swing bridge over the Mokelumne River and also many other jobs where welding is amply involved gives

full confidence in the ability of the qualified welders, civil engineers and contractors to handle and accomplish such work with the best results and economy.

Savings from Welding

The direct saving in metal for a welded bridge, as shown in Table 1, amounts to 189,218 lb. or 25.5% over the semi-welded present trusses equals 33.3%.

The total saving in cost of the whole structure, using welded trusses, amounts to 13.3%.

Table 1 has the comparative weight of each group of members separately and clearly shows in which group arc welding is mostly effective. The gusset plates and filler plates drop to 53%, the welds save 70% as against the riveting. The saving of weight of pivot girder and ring girder needs a special explanation:

By inspection of details of the riveted pivot girder it is seen that a great amount of steel used as construction material in riveted method is purely weight. To that waste weight belong numerous filler plates, web stiffener legs and web flange legs, totaling in weight to 16,280 lb.

Reduced dead load on the pivot girder (780.8 kips for riveted, and 532.2 kips for welded) requires a smaller moment inertia for the welded girder and the weight of girder reduces tremendously. Also, welded girders are effectively proportioned for shear and bending stresses without loss of section due to rivet holes. The same analysis applies to the ring girder.

For average size and loading the saving in favor of welded girders runs about 30% but in this case it goes up to 62.4%, clearly showing the advantage of the arc welding in structural connections.

Conclusion

The method of construction of swing bridges (assembly starts from center of bridge and symmetrically proceeds to ends) allows a free expansion of welded members and by that minimizes locked-up stresses; it is therefore possible to expect the performance of all welded trusses to be not only so satisfactory as the riveted, but better, owing to the fact that there cannot be expected any loosened rivets under traffic, which is important, especially in railway swing bridges. As mentioned before, the absence of rivet heads creates smooth surfaces on the members and increases the life of the bridge by the elimination of many "starting points" of rust around rivet heads and also saves maintenance expenses on painting. Operating and maintenance costs are also reduced because of the lowered weight of trusses and machinery.

Welding Electrodes*

By John W. Miller†

THE manufacturers of Shielded Arc Electrodes have, in the past, been quite secretive about their manufacturing processes. During the last two or three years, however, the secrets have been more or less "debunked," and it has come to the point where most of the coatings are very similar as far as rational analyses of the ingredients are concerned. That is, they are very similar chemically.

Materials of Manufacture

The base material used almost universally in the mild steel electrode is a low carbon, rimmed steel, the carbon ranging from 0.10 to about 0.15%. To confine the range to a narrower limit is quite impractical. The manganese content is 0.40 to 0.60%. Silicon, of course, is practically nil in the rimmed steel, while sulphur and phosphorus are held to 0.04% maximum. However, it is very desirable to keep the sulphur and phosphorus lower than 0.04% if possible.

On wire made from rimmed steel we find on examination of its macro-photograph that there is a characteristic pattern. If this pattern is not in the center it will tend to make the wire burn or melt "lop-sided," and very frequently we will find that on testing a heat of steel, the pattern has actually been "rolled off" center. It makes for a poor grade of welding electrodes.

The testing of the mild steel for acceptance is one about which there has been considerable controversy. If we weld with wire immediately after drawing, and complain to the mill that it is unsatisfactory for welding electrodes, they will generally assert that the pickling process was incorrect or there was not enough lime on the wire. We have adopted the policy of drawing the wire and then emerying off the surface before testing, and if it does not weld under these conditions we reject the material.

Table 1

	Reducing Materials	Slag Forming	Arc Stabilizers	Binder
Cellulose	x			
Ferromanganese	x	x		
Aluminum silicates	..	x		
Magnesium silicates	..	x		
Calcium carbonate	..	x	x	
Titanium dioxide	..	x	x	
Silica	..	x	..	
Iron oxide	..	x	x	
Manganese oxide	..	x	..	
Sodium silicate	..	x	..	x

The original welding electrodes were the so-called "bare wire," which actually contained a stabilizing coating on them; that is, they were so made that they had an iron oxide finish and were then drawn through lime. Very frequently one finds that a wire with that type of finish is satisfactory for welding but with the finish removed it in unsatisfactory. That is one reason for emerying off the coating upon testing of the pilot coils.

Coatings

In the coatings of arc-welding electrodes, there are four general classes of constituents: arc stabilizing compounds, slag forming compounds, deoxidizers and bind-

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* This paper was presented before the Wire Association Convention, held at Cleveland, Ohio, October 1942.

Table 2

A.W.S. Type	Common Designation	Cellulose	Ferro-manganese	Aluminum Silicates	Magnesium Silicates	Lime ($CaCO_3$)	Titanium Dioxide	SiO_2	Iron Oxide	Manganese Oxide	Sodium Silicate
E6030	Deep Groove AC-DC	?	x	x	x	x	?	?	x	?	x
E6020	Flat Fillets AC-DC	?	x	x	x	x	?	?	x	?	x
E6010	Cellulose Reverse Polarity	x	x	..	x	..	x	?	..	..	x
E6012	Straight Polarity All Position	x	x	x	x	x	x*	?	..	..	x

* Rutile.
? Sometimes used.

ing material. The major coating materials are shown in Table 1.

In the molten pool while welding we try to accomplish in an extremely short period of time approximately what the open-hearth furnace does. However, there is one variation. In the majority of the welding electrodes it is impossible to judge from the silica content of the coating whether the slag is acid or basic. The slag obtained from most of the welding electrodes contains about 25 to 35% silicon dioxide. That makes it in the range between the acid and basic processes.

In the coatings of these electrodes we have iron oxide, as either the red or magnetic oxides. It does, to a certain extent, stabilize the arc, but for the practical purpose of shielded arc electrodes the author would not call it an arc stabilizer. It definitely is a slag-forming compound.

Another constituent of coatings is titanium dioxide, which has two functions: first, it does act as an arc stabilizer and is, perhaps, one of the most potent arc stabilizers known. It acts too as a slag-forming compound. However, in very high percentages it imparts some undesirable characteristics to the weld deposit in that it tends to embrittle the weld.

There are also magnesium silicates. These may be in the form of talc or asbestos. Most of the magnesium silicates employed are of the asbestos variety although some talc is still being used, and they act as slag forming compounds.

Besides magnesium silicates, there are the aluminum silicates. These may be in the form of clay or feldspar and are also definitely slag-forming compounds.

Lime or calcium carbonate is both an arc stabilizer and a slag-forming material.

Deoxidizers

Two main deoxidizers are used. The predominant one is ferromanganese, and the majority of the ferromanganese in welding electrodes is of the high carbon variety, that is, 80% manganese, 6.0% carbon. There is some application of low carbon ferromanganese, and also of ferrosilicon as reducing agents. However, high carbon ferromanganese is the main deoxidizer employed, and it has some slag-forming characteristics.

In some slags it is desirable to have considerable manganese and yet to add all of the manganese in the form of ferromanganese may give rather detrimental effects on the weld. Therefore manganese oxide may be added to the coating.

Cellulose is another deoxidizing compound. Almost any of the organic compounds, such as sugar, wood pulp, etc., will act as an organic deoxidizer. However, the odor given off by some organic materials is very undesirable. For instance, wood pulp contains about 50% lignin which gives off an obnoxious odor when burned and alpha cellulose has proved to be the most satisfactory organic, or gas-producing material, and is almost universally employed.

Probably not a single electrode is being manufactured today that does not contain sodium silicate in its formula. Sodium silicate, or water glass, is used primarily as a binder, and it also acts as a slag former.

Materials Used in Various Electrodes

Now as to the materials used in the various electrodes: The cellulose content may vary from 20 to 35% of the total weight of the coating in the cellulose type electrode. A small amount, about 4 or 5%, is used in the straight polarity electrodes. There is very seldom any cellulose in the coating of d.c.-a.c. electrodes. One or two electrodes have been developed that included it, but for practical purposes, we can say there is no cellulose in the a.c.-d.c. type. Some down-hand electrodes include a small proportion of cellulose. However, the majority of down-hand rods have no cellulose in the coating.

Ferromanganese is used as a reducing agent in all types of mild steel coatings.

The principal magnesium silicate used in coatings is asbestos. There is generally a very high percentage of asbestos in the cellulose coating, small amounts in the straight polarity electrode, and small amounts in the a.c.-d.c. electrode.

All coatings have a tendency to crack during the manufacturing operation. The coating actually shrinks upon dehydration of the sodium silicate which is used as a binder, and the tendency for cracking is predominant in the mineral types of coatings. In these types of electrodes it is desirable that a fibrous material such as asbestos be present. Asbestos therefore has proved to be most satisfactory in mineral coatings from a production standpoint.

Clays and feldspars are used almost universally in the three mineral types of coatings (A.W.S. types Nos. E6012, E6020, E6030). Sodium silicate, of course, is used in all the coatings.

Titanium dioxide is always used in the cellulose type of electrodes and in the straight polarity type it may amount to as much as 40 to 55%. Titanium dioxide of pigment purity is a very fine material and is relatively expensive and is used only in coatings containing small amounts of TiO_2. In the straight polarity type, which contains a very high per cent of TiO_2, the manufacturers have substituted rutile in place of TiO_2. Rutile is a brown mineral containing about 95% TiO_2 and which for most practical purposes is just as satisfactory in welding electrodes as pure TiO_2. In fact, some electrodes manufac-

turers have preferred to use rutile in the cellulose coatings. Some a.c.-d.c. electrodes have a small amount of rutile in them as do some down-hand electrodes.

The last materials to be discussed are iron oxide and manganese oxide. Very few of the cellulose coatings have either manganese or iron oxide in them. Some have been manufactured that contained iron oxide, but iron oxide has one bad feature, especially if it is present in very high percentages, in the cellulose coating. The difficulty is that iron oxide tends to cause undercut, and it does slow the melting rate of an electrode. Consequently, less metal is deposited per unit time than if iron oxide were omitted from the coating. Very little, if any, iron oxide is used in the straight polarity electrodes. However in the fillet electrode and deep groove electrodes very high percentages of iron oxide are employed, going up as high as 25% in some instances, and combined with manganese oxide, silica, magnesium oxide and aluminum oxide, these electrodes yield welds having good contour with a slag that is readily removable.

The types of coating materials are illustrated in Table 2.

Avoid Fatigue

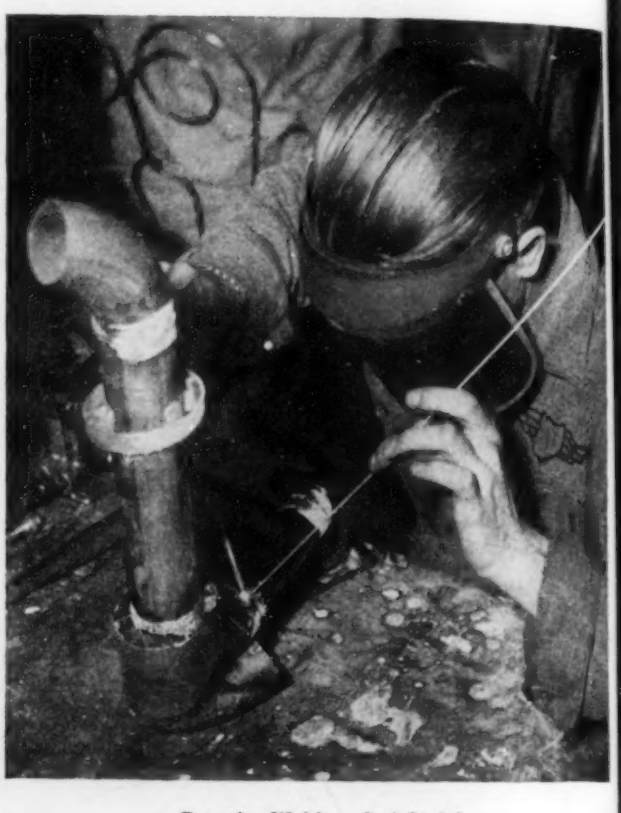
By H. H. Griffith*

SOME welding operators, particularly when they are first learning to weld, complain that their hands and arms become tired after they have been working for a short time. If these operators will analyze the cause of this fatigue they will usually find that it lies in their manner of holding the blowpipe and welding rod—that they are clutching them with a desperate strangle hold.

Muscles will stand just so much strain and then they rebel. If you try to write a letter pinching the pen between your thumb and forefinger, you will find that your hand soon gets cramped—"writer's cramp"—and that you must seek relief by finger exercises.

The same is true in holding a welding or cutting blowpipe. If the welding student keeps a continued strangle hold on it, the muscles of his arm and hand soon get cramped. Naturally he cannot do justice to his job when his arm feels as heavy as lead. The blowpipe should be balanced evenly in the hand, grasped lightly and allowed to swing easily as the weld progresses down the seam.

* Linde Air Products Co.



Grip the Welding Rod Lightly

Balance the Blowpipe in the Hand

Holding the Rod

That tired feeling in the arm holding the welding rod results from the same cause. The rod melts off easily when immersed in the molten puddle beneath the flame. It isn't necessary to *push* the rod into the puddle—pushing doesn't help melt it. If the rod is gripped lightly, the muscles won't tend to become cramped. Many operators find it much less tiring to bend the rod and hold it in the palm of the hand instead of between the thumb and forefinger like a pencil.

Each operator should vary the method of holding the blowpipe and welding rod until he finds the one that is most effortless for him; the best way for one individual may not be the most natural way for another. But no method will be natural if the operator grips the rod and blowpipe, grits his teeth and works with cramped muscles. He will subconsciously be wondering how long he can "hold out," instead of concentrating on good workmanship.

Fractures in Welded Ships

By John Tutin,* D.Sc., M. Inst. N. A.

EDITORIAL NOTE: It is thought this general article, because of certain fundamental concepts, would be interesting to our readers. It is not intended to imply that the solution suggested is the only one or best. Discussion is invited.

A LOCKED-UP stress may be controlled, like friction, in such a way as to be the servant of the engineer. Uncontrolled, it is likely to be his enemy. For example, locked-up stresses are consciously or subconsciously employed, under control, in many familiar devices such as a nut and bolt, a rivet, a loaded spring, a shrink fit, a braced wireless mast and numerous weapons of war, historic, prehistoric and twentieth century. Out of control, locked-up stresses may cause the total failure of steel castings, bridges, ships, aircraft and other structures, large or small. The recent failure of the S.S. *Schenectady* is by no means an isolated case.

The phrase "locked-up stress" is somewhat crude and many alternatives are used from time to time such as "residual stress," "internal stress," "contraction stress," "thermal stress" and others. On balance, the phrase "locked-up stress" is probably the most convenient description for general purposes.

Definition.—A locked-up stress is a stress which exists in an unloaded structure, that is, a structure not acted upon by any external forces.

It follows that in a loaded structure, the locked-up stress at any point will be that proportion of the total stress at that point which does not vary with the load.

Fundamental principles. At any section through an unloaded structure containing locked-up stresses, the following conditions of equilibrium must be satisfied:

1. The algebraic sum of the forces acting on any one side of the section is zero.
2. The moment of resistance of the section is zero.

Further rules can now be written down as follows:

3. It is impossible to have a structure containing a single locked-up stress.
4. It is impossible to have a structure containing only two locked-up stresses. The minimum conditions for equilibrium are either three locked-up stresses or two locked-up stresses in association with one locked-up moment.
5. In a pin-jointed structure, locked-up stresses cannot exist in any given member unless a line drawn in any direction intersecting that member also intersects at least two other members of the structure, in which case the nature of the stresses taken in sequence can only be of alternating signs: $(+\ -\ +)$ or $(-\ +\ -)$.
6. In a bracketed structure, locked-up stresses cannot exist in any given member unless a straight line drawn in any direction through that member intersects at least one other member, in which case the stresses must be of opposite sign, with a moment or moments intervening: $(+, \mathrm{M}, -)$.

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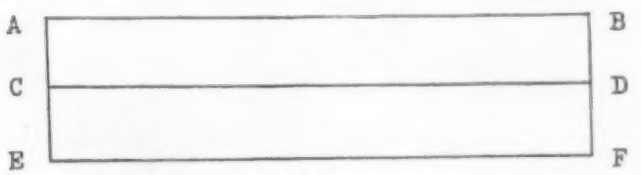


Fig. 1

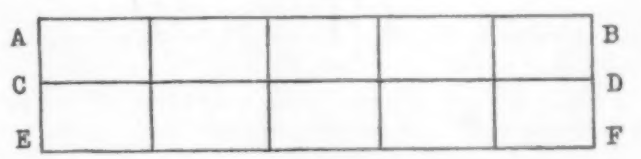


Fig. 2

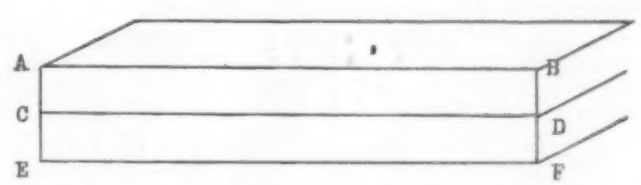


Fig. 3

In relation to the foregoing principles, let us consider the simple but instructive example shown in Fig. 1. AB, CD and EF represent horizontal members of a structure, connected by vertical members AE and BF. Locked-up stresses may be set up in this structure by introducing, for example, a tension in AB. This will induce a compression in CD and a tension in EF. For the benefit of those who find it difficult to "visualize" the conditions indicated in Fig. 1 a familiar example is a violin. The string corresponds to AB, the body of the case to CD and the back of the case to EF. Tightening the strings (tension) produces a tendency to crush the body (compression) and rupture the back (tension).

If the structure is symmetrical, and the force in AB is P, the force in EF is also P, and in CD it is $2P$. If in such a case the cross sections AB, CD and EF are equal, the compressive stress in CD will be double the tensile stress in AB and EF.

Alternatively, a tension introduced into CD will induce compressions in AB and EF.

The stresses in AB, CD and EF produce bending moments and shearing forces in AE and BF.

For a given amount of contraction or expansion in AB, the stresses developed in the horizontal members depend solely on the stiffness of the vertical members, that is, the more rigid we make the vertical members the greater the stresses produced by a specified amount of thermal contraction or expansion in AB.

The introduction of additional vertical members as indicated in Fig. 2 is equivalent, for a specified amount of contraction or expansion in *AB*, to an increase in the effective rigidity of the vertical members and will therefore result in increased stresses in the horizontal members.

We now proceed from Fig. 2 to Fig. 3 by substituting horizontal surfaces in lieu of *AB* and *EF*, and vertical surfaces in lieu of *CD*, *AE* and *BF*, giving us a hollow closed box. The same general principles still apply: Tension set up in *AB* will produce an induced compression in *CD* and an induced tension in *EF* and the rigidity necessary to transmit and maintain these stresses in equilibrium is largely provided by the resistance to shear in the sides.

It will be seen that Fig. 3 is our old friend, the box-type ship.

In the case of a two-deck ship, *AB* corresponds to the upper deck, *CD* to the second deck and *EF* to the double bottom.

The sequence of operations is usually such that the final welding of the upper deck is liable to set up a thermal contraction. If this occurs, a compression is induced in the second deck. These stresses may be serious even if no further stresses come on the structure, but when combined with the normal bending stresses in a seaway or even in still water, it is not surprising that failure may ensue:

1. The locked-up and service stresses in the upper deck in the hogging condition are additive.
2. The locked-up compressive stress in the second deck will prevent that deck from carrying tensile stress in the hogging condition, until sufficient bending has taken place to bring the plating into tension.
3. The effect of this induced compression in the second deck will be to increase the tensile stress which the upper deck must carry. In fact, until the compression in the second deck has been reduced to zero, the vessel must be regarded as a single deck ship from the point of view of its resistance to longitudinal bending.
4. The induced compression in the second deck will tend to produce buckling and, if buckling occurs, it is likely to coincide in longitudinal position with the incidence of locked-up tension in the deck above. This is confirmed by the observed facts. Incidentally the marked tendency of the ends of a welded ship to lift during construction is of special interest as indicating the magnitude and nature of the forces at work.
5. Stress transmission between the upper deck and the side shell will augment the load carried by the shear strake-stringer connection, the welding of which calls for special consideration in relation to peak loads in the hogging condition.
6. The reaction tension in the double bottom (keel, bilge, and tank top) will, in a normal case, be considerably less than the action tension in the upper deck (equating moments about the second deck). Therefore the double bottom may be expected to function with reasonable efficiency in both the hogging and the sagging condition.
7. The upper deck, in the sagging condition, cannot develop a compression stress until the tension therein has been reduced to zero.
8. The neutral axis in a seaway will be in a state of vertical oscillation relatively to the keel, the amplitude of oscillation being largely governed by the magnitude of the action and reaction stresses.
9. The hull deflection for moderate conditions of loading, will, if locked-up stresses are present, be greater than anticipated, due to the reduction in effective moment of resistance. On the other hand, the deflection, associated with incipient local fracture will be much less than anticipated owing to the reduction in the margin of safety. This, in a sense, is equivalent to a "reduction in flexibility."
10. At the changeover point between hogging and sagging, in association with a sudden movement of the neutral axis, there will be a sudden increase in the time rate of change of hull deflection. This "snap" effect is important in relation to the impact values of the plating and weld metal.

The *Schenectady* broke in two in still water with a calculated maximum stress of $4^1/_2$ tons per square inch, indicating the presence of locked-up stresses of considerable magnitude, even after making full allowance for other factors. Less spectacular but equally important fractures have occurred from time to time, on the stocks, in still water and at sea. The official report on the *Schenectady* states:

"In the opinion of the Committee the failure of the hull structure of the *Schenectady* was due to a combination of circumstances. The principal cause was an accumulation of an abnormal amount of internal stress locked into the structure by the processes of construction."

Calculations made for the "Liberty" type of all-welded ship show that the effect of an induced compression in the second deck may by itself cause an increase in the nominal service stress on the upper deck of the order of 30% apart altogether from the numerical values of locked-up stresses, and stress raisers. Incidentally, an inspection of a major fracture on one of these vessels revealed that the fracture, which started in the upper deck under tension, entered the second deck in way of a pronounced transverse buckle, the crack running significantly up the trough of the buckle.

Nevertheless electric welding of ships has come to stay. Apart from its overwhelming importance today as a means of obtaining a vast output of new tonnage, it will inevitably play an increasingly important part in the future of shipbuilding. It is a more agreeable occupation than riveting, and it offers a substantial reduction in steel weight compared with riveting. This reduction can only be fully achieved by removing any element of uncertainty and this calls for:

1. Methods of preventing major locked-up stresses in welded ships under construction.
2. Methods of removing major lock-up stresses in existing welded ships.

The phrase "major locked-up stresses" is used deliberately, because locked-up stresses are not unknown in riveted ships, and are equally undesirable, but as a rule are better understood and more easily remedied.

In new ships the prevention of locked-up stresses should primarily be the responsibility of the structural designer, and only in a secondary capacity should it be the concern of the "outside" personnel.

Correct design should make it immaterial whether or not a given welding procedure or type of welding machine may tend to cause locked-up stresses. There are obvious ways of ensuring this without impairing in any way the efficiency of the structure against service loads or impeding production. For example, a locked-up stress in a welded deck can be prevented by introducing a transverse line or lines of rivets. Similarly in the shell, tank top or double bottom. If there is any doubt, a complete transverse belt of riveting can be arranged right across the ship at a convenient station. Since in most cases the action stress resides in the upper deck, it might prove a sufficient precaution to confine this band of riveting to the upper half of the structure. It would obviously be wrong to regard such an arrangement as an "expansion joint." It is essentially a means of stress re-

lieving and the effect is therefore equivalent to large-scale annealing. The effect of a completely riveted shear strake-stringer connection would be very similar, and should be adequate if confined to the amidship half length. Any such riveting should always be completed last. At least one important continental shipyard employs the riveted belt amidships on all-welded vessels including warships, this particular shipyard having adopted this standard construction as a result of practical experience with welded ships over a number of years.

Although the basic principles dealt with in this memorandum are essentially simple and indeed elementary, they have not so far been set down, and there is, of course a wide field of detailed technical application, not only to ships but to welded structures of all types.

Discussion by Ralph E. Spaulding,* M. Am. Soc. C.E.

The writer has read with much interest the article by Dr. John Tutin on "Fractures in Welded Ships." The common-sense view taken by Dr. Tutin that locked-in stresses are always present in any structure and that the correct approach to the problem is to make them work for us rather than against us is very refreshing.

The principal reason why locked-in stresses are usually present in welded structures, as ordinarily built, is that the designing engineer follows automatically the same general type of design heretofore used in a similar riveted structure. That is to say he uses, in general, the same type of sections similarly placed with respect to one another, the only substantial difference being that the joints are welded instead of riveted. As a result the structure so designed not only lacks the full economy obtainable by welded design but also there are often developed (unnecessarily) objectionable locked-in stresses which impair the efficiency, or maximum load-carrying capacity, of the structure.

Figure 1: In a welded assembly of plates, as illustrated in Fig. 1, it is known that there exists in the weld metal and a part of the parent metal adjacent thereto, along the axis of the weld (as along the line *AB*), tension up to the full elastic limit of the metal, this force being balanced by compression of the same total amount (but of much lesser unit magnitude) in the remainder of the plate area. The shrinkage in the welded seams also creates a "pull" transverse to the line *AB*. If the plates are free to "creep" transverse to the line *AB* there will remain no appreciable locked-in tensile stress transverse to *AB*, either in the weld metal or in the plates (except at the transverse welded joints, as *CD*). But even where the plates are quite rigidly held by very heavy transverse frames, this residual transverse stress is usually negligible both in its magnitude and in its effect upon the factor of safety of the structure.

Figure 2: In the ordinary type of welded steel ship the stress transmission between the side shell and the upper deck is carried principally by the weld metal illustrated in one of the two sketches of Fig. 2. In Fig. 2 the cross-hatched area at the weld is (before the structure has been subjected to working loads) under tension, along the joint, up to the full elastic limit of the metal; whereas, the metal in the plates on either side of the weld is under compression (to balance the total shrinkage tension). When this welded joint is put to work, as in hogging action, the weld metal is stressed beyond its elastic limit and at the same time it is being subjected to heavy shearing stresses to transfer stress from the side shell to the deck—not a desirable condition. Yet the weld metal can, and does, so yield in a very uniform manner unless this uniform yielding is interrupted by adverse temperature conditions, improper characteristics of the metal, "notch effects," or "stress eddies" brought in by intersecting transverse welded seams, improperly welded, in which case a crack may occur, sometimes with unpleasant results. Bear in mind that the weld metal is under tension up to its elastic limit and that the metal on either side of the weld is under compression so that if the weld cracks under added tension the stress on either side of the weld changes suddenly from compression to tension which helps the fracture along.

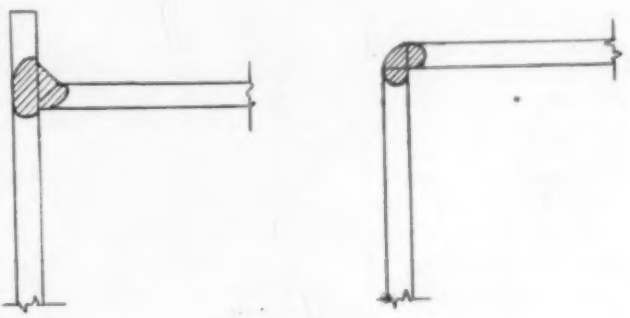


Fig. 2

Nothing is to be gained, but on the contrary, much harm done the welding industry, by failing to point out that this condition exists. The reasonable thing would seem to work toward the elimination of the undesirable condition.

This locked-in stress (as above described and as illustrated in Fig. 2) can be quite effectively eliminated by hot

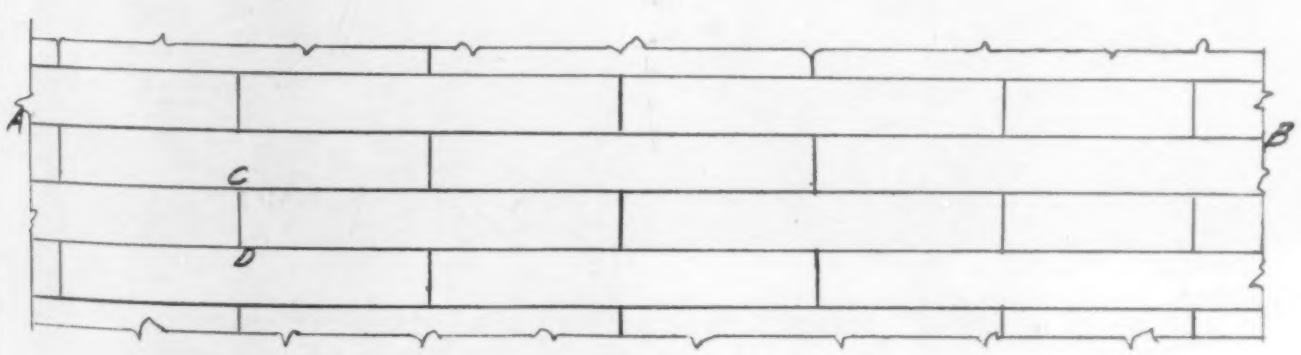


Fig. 1

*The Aetna Steel Company, Jacksonville, Florida.

peening while the weld area is cooling from its original weld heat. While locked-in stresses can be, and have successfully been, eliminated by this hot peening procedure it adds somewhat to the cost and is seldom the most satisfactory approach to the problem. It is usually cheaper and more desirable to design the structure and the welding of it so that, as Dr. Tutin suggests, the locked-in stresses, if not working with us, at least will not work against us.

Now, referring again to Fig. 1, it is seen that the condition along the longitudinal welded seams is similar, although usually less critical, to the condition in the welded joints of Fig. 2, and when the plate assembly is subjected to longitudinal tension the same type of difficulties occur. Therefore, it would seem evident that in any welded plate assembly the long continuously welded seams (as *AB* of Fig. 1) should be placed at right angles to the principal tensile stress. Then the difficulties described under Fig. 2 would largely disappear.

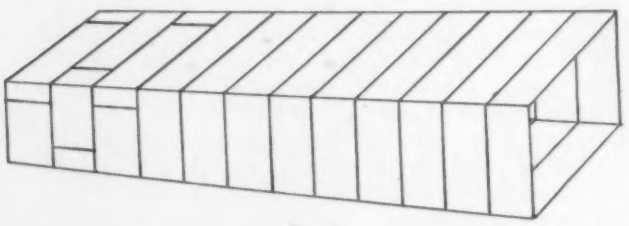


Fig. 3

Figure 3: To illustrate the above refer to Fig. 3 (corresponding to Fig. 3 of Dr. Tutin's article).

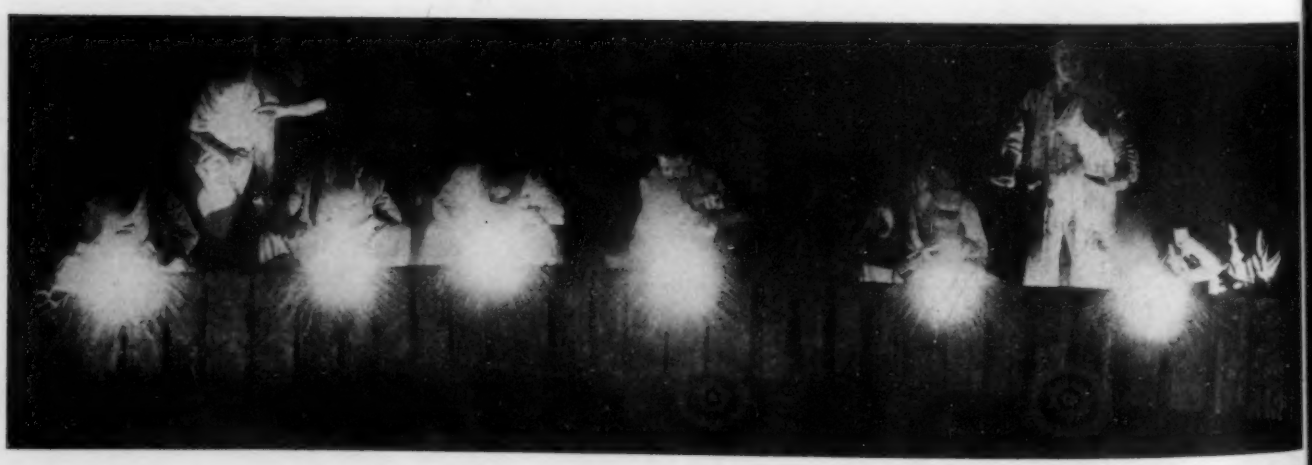
Assume this box (hull) built of continuous strips as indicated at the right-hand end of Fig. 3 and welded all the way around. Start with the center strip and proceed toward both ends, allowing the fore and aft shrinkage to take place freely before the strip is frozen to any longitudinal member. The final result would be a slight shrinkage at the periphery with a more substantial fore and aft shrinkage of the plating. As to the periphery, the welded joints are under tension with the plates between the joints in compensating compression parallel with the joints. While the peripheral seams would (under working loads) be subjected to shearing stresses, the magnitude of these would not be great enough to approach that of the stresses in the shear strake-stringer connection as illustrated in Fig. 2.

Longitudinally (fore and aft) there would be practically no locked-in stresses in the plating whatever, and the metal and welds would be free to take stresses due to hogging or sagging without interference.

Actually there would be longitudinal seams at the ends of each plate as roughly indicated at the left-hand end of Fig. 3. These short, and discontinuous, fore and aft seams, welded first (ahead of the peripheral seams), would make no substantial change in the resultant final working stresses as described above.

The writer has suggested, above, two solutions for the elimination of the danger which often exists at the junction of the side plating with the deck:

1. Take out the locked-in stress by hot peening.
2. Eliminate the fore and aft welded seam at that point entirely (at least as to the straight center section of the hull where hogging and sagging stresses are greatest) by the logical process of designing properly for a welded structure, as indicated in Fig. 3, instead of blindly following riveted design and trying to adapt welding to it.



Courtesy Ernesto Trivellato, Sao Paulo, Brazil.

Repairing of a Caterpillar Tractor Tread

The repair of this caterpillar tractor tread was completed at night, in 4 hrs. by six welders working at the same time.

Joe needs the Long Distance lines tonight

He has a promotion to report. Or a week-end leave coming up. Or it's his mother's birthday.

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THE AMERICAN WELDING SOCIETY

ACTIVITIES – RELATED EVENTS

OFFICE OF WAR INFORMATION

In an effort to improve the quality and quantity of shipyard welding work, the United States Maritime Commission recently announced it had accepted the offer of The Lincoln Electric Co., of Cleveland, Ohio, to train and instruct welders in shipyards engaged in Maritime Commission work.

The company will furnish training facilities in Cleveland, including the necessary equipment and staff for training supervisors and teachers who will operate the various welding schools in Maritime Commission yards. The company will also provide the Commission with welding engineers to assist the yards in raising their welding standards, and will submit monthly reports to the Commission showing progress being made by the new service.

METAL DIRECTORY

This is the ninth edition of the *Standard Metal Directory*, the previous one having been published in 1940. Wartime changes made it necessary to completely revise the old edition. All information has been brought up to date. The 1943 edition contains over 700 pages.

The directory is divided into four special sections embracing: Iron and Steel Plants; Ferrous and Non-ferrous Metal Foundries; Metal Rolling Mills; Smelters and Refiners of Non-ferrous Metals.

In these lists will be found more than 7000 detailed reports on steel mills, ferrous and non-ferrous metal foundries, metal smelters, metal rolling mills and non-ferrous metal plants in the United States and Canada, arranged geographically and alphabetically. These reports give the name of the company, its capitalization, when organized, the location of its plants, the address of the main office, company officers, purchasing agent, sales manager, plant equipment products manufactured, and type of raw and semi-finished materials consumed.

In addition to these four main sections, there are special lists of manufacturers and distributors of pig iron, ores and alloys; smelters and refiners of new and old metals; storage battery manufacturers; solder manufacturers; sheet metal stamping works; galvanizers; die-casting plants; secondary metal smelters; railroad purchasing agents; copper, lead and zinc brands; scrap iron and scrap metal dealers in the United States and Canada.

Three new lists have been added to round out the coverage of the directory; one of fabricators of structural steel, bars, sheets, tanks, plates and boilers; the second lists airplane manufacturers and the third, merchants who specialize in used structural steel products.

The *Standard Metal Directory* is an excellent and valuable reference for the steel and metal industries and is of special value to purchasing agents and sales managers. It is published by the Atlas Publishing Co., 133 West 21st St., New York, and sells for $10 a copy.

OFFICE OF WAR INFORMATION

The United States Maritime Commission recently awarded four shipyards high production honors for improving past records in constructing and repairing ships to meet the wartime demands of the Nation.

Shipyards winning the Maritime "M" Production Award are Walter Butler Shipbuilders, Inc., of St. Paul, Minn., for achievement in coastal cargo ship construction, and Moore Dry Dock Co. of Oakland, Calif., for outstanding achievement in construction, conversion and repair of merchant vessels.

North Carolina Shipbuilding Co. of Wilmington, N. C., and Permanente Shipyard, Richmond yards Nos. 1 and 2, Richmond, Calif., were awarded the Maritime Merit Eagle Pennant for continued outstanding production achievement in delivering merchant vessels from all their ship ways.

Shipyards who were previously awarded the Maritime "M" were awarded Gold Stars to be added to their "M" Pennant, signifying continued production achievement. These shipyards are: Delta Shipbuilding Co., Inc., New Orleans, La.; Houston Shipbuilding Corp., Houston, Tex.

ARMY-NAVY "E" AWARD TO BASTIAN-BLESSING

For great achievement in the production of war equipment, the Army-Navy "E" Award has been conferred upon the men and women of the Chicago plant of The Bastian-Blessing Company. The formal ceremony was held on the spacious lawns of the Company's modern new plant at 4201 Peterson Ave., on July 26th.

Immense quantities of fuses for 75-mm. guns, parts for warplanes, valves for fire extinguishers, fliers' oxygen lines and life rafts and other precision equipment have been turned out, on an around-the-clock schedule, by the Bastian-Blessing factory, which is one of the industrial show places of the Chicago area. For more than a year the facilities of this Company, one of the country's largest manufacturers of cutting and welding apparatus, equipment for controlling liquefied petroleum gas, fountain-luncheonettes, frosted food and ice cream cabinets, carbonators and counter ice cream freezers, have been devoted entirely to war production.

AIRCO "M" AWARD EMPHASIZES IMPORTANCE OF WELDING

In a ceremony highlighting the vital importance of oxyacetylene flame cutting and arc welding to America's shipbuilding program, Rear Admiral Emory S. Land today gave a highly encouraging picture in the progress made in building our Victory fleet. The hard-hitting Chairman of the U. S. Maritime Commission said:

"Credit has been given the United States Maritime Commission for the greatest shipbuilding program the world has ever known. We gladly share such honors with the people on the home front. . . . New methods of welding and cutting, of making shapes and forming steel, which the workers of this company have developed, have greatly accelerated our ship production program. The building of a Liberty ship requires the cutting of approximately 3500 tons of steel plates. Without the cutting tools and gases such as this company furnishes, this process would indeed be a slow one. Instead of building Liberty ships with an average of approximately 600,000 man-hours, we might have to double that figure. It would mean we would build only half the ships we are building today. We could not meet our President's directive for ships and more ships unless we received the all-out cooperation of plants like yours. These factories of yours are a battlefront. Through your aid we are winning the battle of production. . . . I have been asked many times what it was that enabled us to meet our production goals. I have always stressed the Siamese Twins of shipbuilding speed—prefabrication and welding. These two things have greatly speeded production. Welding also has gone farther than that. It has increased the speed of our ships. It has reduced their weight and enabled them to carry more of the cargoes our fighting men need. It means that we deliver to the fighting front more bombs, more guns, more tanks and more planes because we have saved weight and have added speed through welding. . . . I wish I had time to tell you of the valued service which the merchant seamen of this Nation are giving to the war effort. At the office in Washington one frequently sees merchant seamen with the Combat Bar which is given by the War Shipping Administra-

For Champion Production·····CHOOSE

When exceptionally high quality of welds is required such as in the case of the boiler drums for our Merchant Fleet, Champion Red Devil "75" electrodes are usually employed. They produce sound, clean welds of high quality. The alloying elements in the coating impart high resistance to fatigue and heat.

"Be sure it's a stub before it's thrown away."

Photo Shows Champion Red Devil "75" electrode being used on a marine boiler drum in a large mid-west plant.

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tion to seamen who have been torpedoed or seen other action. On some of those bars there are five or six silver stars, indicating that the proud wearer has met the enemy that many times. Each one of those stars means that the enemy has blasted his ship. . . . They will not fail us and they issue to every factory and shipyard worker a challenge—a challenge of teamwork. 'You build them—We'll sail them'—these ships for Victory."

The Award of the Maritime "M" and Victory Flag for outstanding production achievement by Admiral Land was extended not only to Air Reduction but also to its seven subsidiary companies, to all employees and to their 159 plants, factories and laboratories located throughout the United States

Twenty-two employees, chosen to represent the 8016 men and women working in 159 plants, all through the country, were present to receive the Maritime Commission Labor Merit Badge. Walter J. Strawn, of Air Reduction's Oxygen Plant in Bethlehem, upon receiving his badge from Admiral Land, pledged continued all-out support from every employee in every plant, in furnishing oxygen, acetylene and the extensive welding and cutting equipment essential to greater shipbuilding achievement.

SHIP WELDING HANDBOOK

Ship Welding Handbook, by Martin J. Coen. Publication date, July 24, 1943. 505 pages, 261 illustrations, 32 tables, $3.50.

Today, national emphasis is on ships. Shipyard emphasis, in turn, is on welders and welding, not merely good welders and good welding, but expert welders and expert welding.

The *Ship Welding Handbook* is most timely. The author had the beginner in mind, in preparing this handbook, as much as the ordinarily good welder who wants to become an expert.

The author is a Welding Engineer of the United States Navy Department. Eleven years of study and field practice, and the resulting experience, are behind the simple, direct language that makes clear the technical details of welding and its related activities.

WAR PRODUCTION BOARD

Resistance Welding Equipment

The fulfillment of requirements for the defense of the United States has created a shortage in the supply of certain materials and facilities used in the manufacture of resistance welding equipment for defense, for private account and for export; and the following order is deemed necessary and appropriate in the public interest and to promote the national defense:

Authorization of Purchase Orders Required. (1) On and after July 27, 1943, no manufacturer or dealer shall accept an order for, or deliver any new resistance welding equipment unless the order or delivery is specifically authorized by the War Production Board on Form WPB-2752. Application for an authorization, and for a preference rating if none has been previously assigned, is to be made by the purchaser by filing Form WPB-2752, in duplicate, with the War Production Board as explained in the instructions which accompany the form. The delivery restrictions of this paragraph (c) (1) do not apply to orders received prior to July 27, 1943.

The provisions of above paragraph shall not apply to (i) any order of $200 or less for resistance welding equipment; (ii) any order for resistance welding electrodes; (iii) any order for resistance welding equipment for direct use by the Army, Navy, Maritime Commission or War Shipping Administration or for incorporation in or attachment to any resistance welding equipment to be used directly by such agencies; (iv) any order bearing a preference rating assigned under Preference Rating Order P-19-h; *Provided That*, notwithstanding paragraph (g) (4) of Priorities Regulation 3, the certificate applying or extending such rating shall state the source of the rating; or (v) any order placed by a manufacturer of, or dealer in, resistance welding equipment

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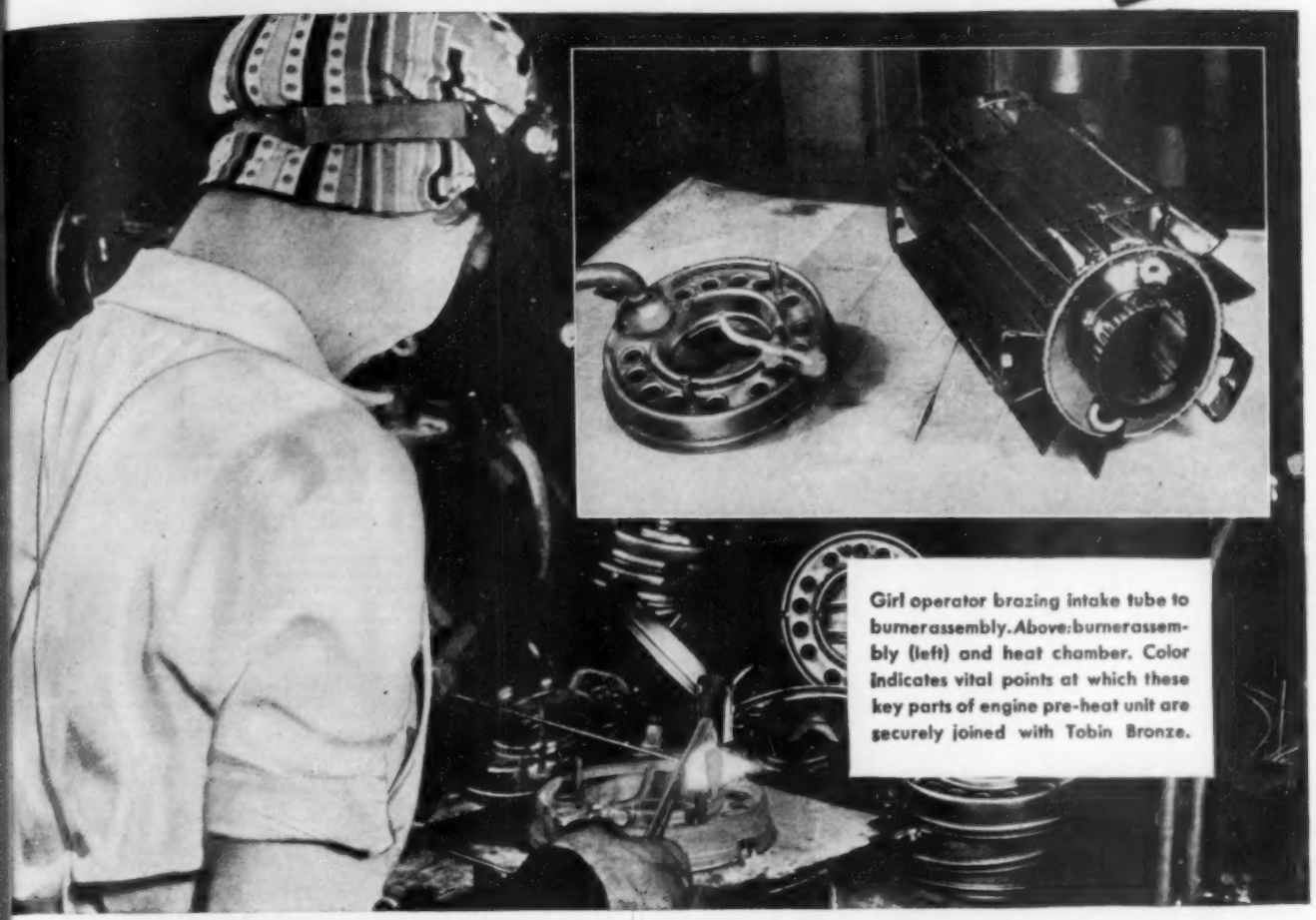
BRONZE WELDED PRE-HEAT UNITS



Girl operator brazing intake tube to burnerassembly. Above: burnerassembly (left) and heat chamber. Color indicates vital points at which these key parts of engine pre-heat unit are securely joined with Tobin Bronze.

SAVE MINUTES GETTING TRUCKS AND TANKS IN ACTION

That's why the Evans Engine Pre-heater, produced by Evans Products Company, is widely acclaimed in combat areas. For, with this gasoline-air heater, automotive and tank engine parts and oil are held near operating temperatures at all times. Because the "warm up" period is reduced, trucks and tanks have quick get-away in spite of cold. Also engines suffer less wear . . . last considerably longer.

Two key parts of this important engine pre-heater—the burner assembly and heat chamber—are joined easily, surely, by production welding with 3/32" dia. Tobin Bronze Welding Rod.

Here—as in a wide variety of other production and repair welding applications—Tobin Bronze* was the choice because of its uniform composition, good strength and toughness, and easy application. This general-purpose rod is especially useful for bronze welding cast and malleable iron, steel, copper and its alloys, and the nickel alloys. Tobin Bronze is also widely used for reclaiming fractured cylinders and other locomotive parts, cracked engine blocks, damaged machinery and broken industrial equipment.

For the latest information on welding with Tobin Bronze—and all the Anaconda Copper Alloy Welding Rods—write for Publication B-13. 4787

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Anaconda Welding Rods

LUKENWELD PRESIDENT

C. L. Huston, Jr., assistant to the President and member of the Board of Directors of Lukens Steel Co., has been named president of Lukenweld, Inc., effective immediately, according to an announcement by Robert W. Wolcott, president of Lukens and Chairman of the Board of its Lukenweld subsidiary.

Mr. Huston succeeds Everett Chapman who resigned to establish his own business as a consulting engineer. Before his appointment as assistant to the President of Lukens, Mr. Huston had served the company as director of personnel relations, a position he assumed in September 1939.

Lukenweld, Inc., is an important fabricator of welded structures used in marine and rail transportation and for industrial machinery. The company is now engaged in production of such vital war materials as component parts for Diesel engines and propulsion equipment for destroyers and escort vessels.

Prior to going with Lukens, Mr. Huston served ten years with the American Rolling Mill Co. in Middletown, Ohio. With Armco, he spent five years in the Metallurgical Department and five years on the company's personnel relations staff in public relations, employment and training work. At the time of his resignation to join Lukens, he was Assistant Staff Supervisor of Employment for Rolling Mill in charge of introductory training.

Mr. Huston is a member of the Industrial Relations Committee of the American Iron & Steel Institute and a member of the Association of Iron & Steel Engineers. He has recently served as a special mediating officer for the National War Labor Board.

Mr. Huston was born in Coatesville in May 1906, and attended grade school there and graduated from the Hill School in Pottstown, Pa., in 1924. He graduated from Princeton University with a degree of Bachelor of Science in 1928 and spent the following year doing special studies in metallurgy at Massachusetts Institute of Technology, going from there to the American Rolling Mill Co.

FABRICATED PIPING

Fabricated piping, oddly enough, has many more applications than most business men realize. In fact, there is scarcely anything tubular in the world that does not come under the head of fabricated piping.

Invasion barges and power plants, tank factories and paper mills, battle ships and filling stations . . . they all require fabricated piping—to convey gas, air, steam, oil and chemicals, or for construction purposes.

A highly interestingly illustrated book has just been issued by the Flori Pipe Co., of St. Louis, a company long in the business of pipe fabrication. It covers the various ways in which piping can be used and lists present market prices for every conceivable kind and size of fabricated piping.

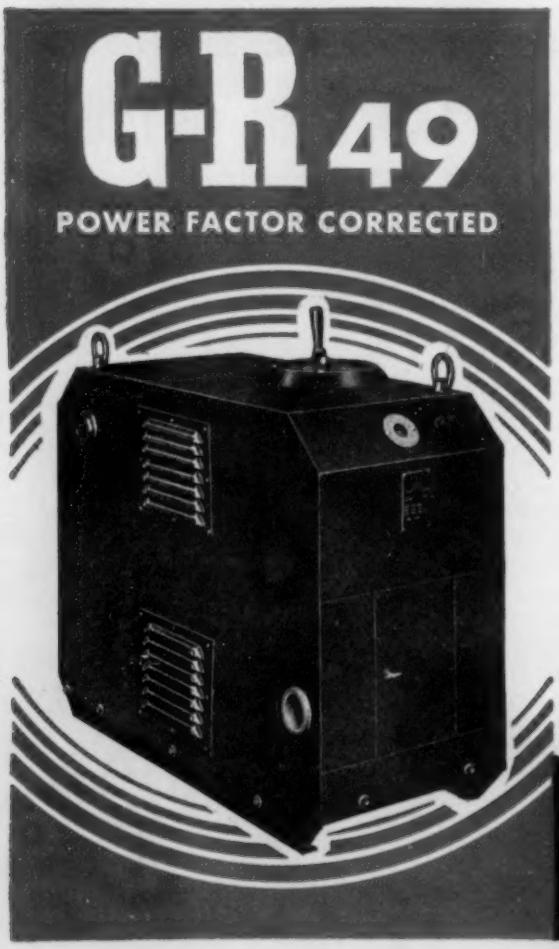
This small book will be useful to you for handy reference. If you would like to have a copy, write to Dept. A-13 of the Flori Pipe Co., St. Louis, Mo. They will be glad to mail you one without charge.

HAMMOND RECEIVES ARMY-NAVY "E"

The Army-Navy "E" was awarded to employees of The Hammond Iron Works in a colorful ceremony held in Warren, Pa., on July 20th. Col. Herbert D. Vogel, District Engineer in the U. S. Engineer Office at Pittsburgh, presented the flag to H. D. Kopf, company president, and to E. D. Huber, local union president. R. Y. Kopf referred to some of the war activities of the company which fabricates and erects all types of process and storage vessels for munitions, chemical, petroleum and allied industries. The company has constructed many important storage facilities at Army and Navy bases located outside continental United States.

BRAZING NEWS

Low Temperature Brazing News gives a broad picture of what Sil-Fos and Easy-Flo are doing in war production. The Handy & Harman Co., 82 Fulton St., New York will be glad to mail a copy to anyone sending in a request.



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Two open circuit voltages greatly widen its field of welding applications. G-R 49 is supplied with contactor having push-button control for power supply; handles rod from ⅛-inch to ⅜-inch with only a twist of the hand wheel. Welding range 100-675 amperes. Get the whole story from your nearest G-R Distributor.

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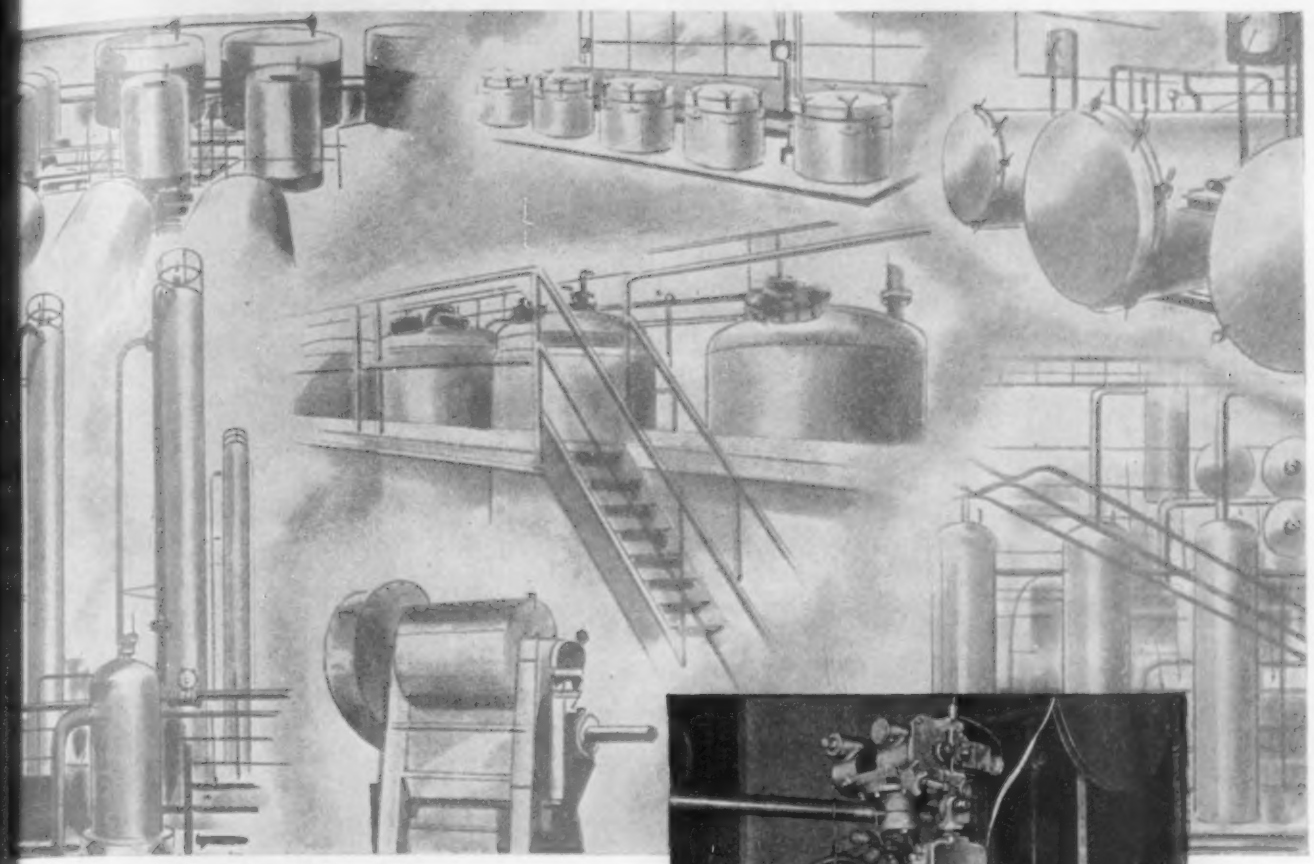

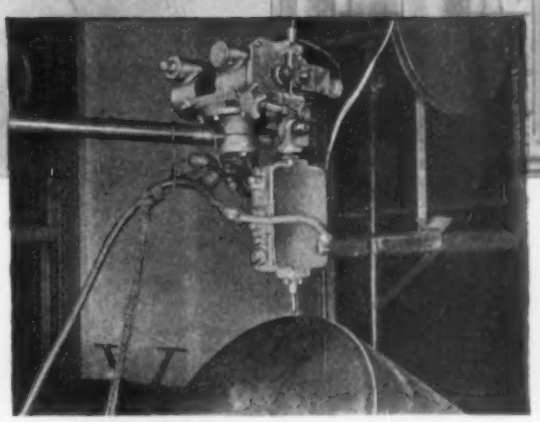

The Steel Cylinders of War and Peace are made Faster, Better, Cheaper by Unamatic

As shells and bombs, as tanks and pipelines for petroleum, as vats and kettles for food, as digestors and stills, kilns and dryers in the chemical industries, steel cylinders are the very basis of the war effort. Steel cylinders are now saving the modern civilization which they helped to build, and soon they will play again an even bigger part in the progress of peace.

Unamatic shielded arc welding is two to three times faster than manual on circumferential and longitudinal welding of steel cylinders of any size. In pressure vessels the Unamatic process insures the highest attainable uniformity and strength of the weldment while at the same time reducing the cost.

Many of the newest technologies are based upon cylindrical steel containers, especially pressure vessels, and will continue to expand in scale after the war. The Unamatic process has already revolutionized fabrication methods in this field, and under normal competitive conditions its outstanding advantages over any other method will be inescapable.

If you are seeking definite, specific, usable ideas for industrial planning that starts now, write for detailed information about the Unamatic process and equipment.

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EDGE MOOR BULLETIN

Eliminating all non-essentials, this new bulletin gives a quick working knowledge of the capabilities of Edge Moor Shops to produce equipment in the variety of designs and in the wide range of metals necessary for successful operation under corrosion, temperature and pressure conditions encountered in the chemical industries. Copy on request, Edge Moor Iron Works, Inc., Edge Moor, Del.

POCKET MANUAL OF ARC WELDING

The *Pocket Manual of Arc Welding* is a handy 128-page book particularly designed for the welding operator and supervisor as a source of quick information on welding data.

The facts are so outlined as to be of material assistance to one very large class of users—the group who are no longer students and who are not engineers—the practical welding operator and supervisor.

This handy-size volume, at the low price of 50¢, is a *must* for any welding operator. It contains over 70 diagrams, tables and illustrations. The Manual is published by The Industrial Publishing Co., 812 Huron Road, Cleveland, Ohio.

VICTOR HOUSE ORGAN

The Victor Equipment Co., 844-54 Folsom St., San Francisco, have come out with a new house organ entitled "Victor Torch Talk." In the words of Mr. E. L. Mathy, Vice-President of the Company, its objectives are broad:

"We want to stimulate maximum cooperation among a great number of new employees. . . .

"We want to tell our co-workers how the products they manufacture help in our war effort. . . .

"We want to let them know something about their industry and about their company. . . .

"We want to acquaint them with such Government orders as will affect their lives. . . .

"And—we want to try to do all of these things without paternalism or preaching."

HYDRAULIC ELEVATING TABLE

Improvements that further extend the adaptability of the Lyon Hydraulic Elevating Table to many uses have been incorporated by Lyon-Raymond Corp., 1271 Madison St., Greene, N. Y., on the model that has been marketed for the last three years and which has proved widely popular for: (1) handling dies; (2) locating work in convenient positions for welding; (3) transferring heavy parts from one level to another level; (4) supporting overhanging work; (5) bringing work to convenient heights for the operator; and (6) operations where easy foot-actuated elevation is advantageous.

Special tables can also be furnished with any of these features and others, as well as of different capacities, sizes of table top and lowered and elevated heights.

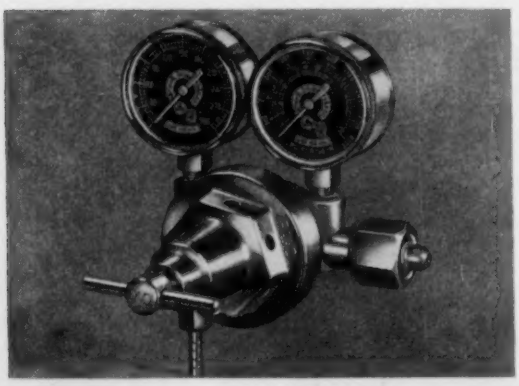
LOUREE PROMOTED

The Ingalls Iron Works Co. of Birmingham wishes to announce the appointment of C. Harold Louree as District Manager of the New Orleans Sales Office.

Mr. Louree was with the Austin Co. at Oklahoma City as Designing Engineer and previous to that he was with the Patterson Steel Company of Tulsa as Sales Engineer. He is an associate member of the American Society of Civil Engineers and the AMERICAN WELDING SOCIETY.

GORDON APPOINTED GENERAL MANAGER

Mr. John D. Gordon a member of the Board of Directors of the AMERICAN WELDING SOCIETY has been appointed General Manager of the Pomona Pump Division of the Joshua Hendy Iron Works of Sunnyvale, Calif., with headquarters at Pomona, Calif. The Joshua Hendy



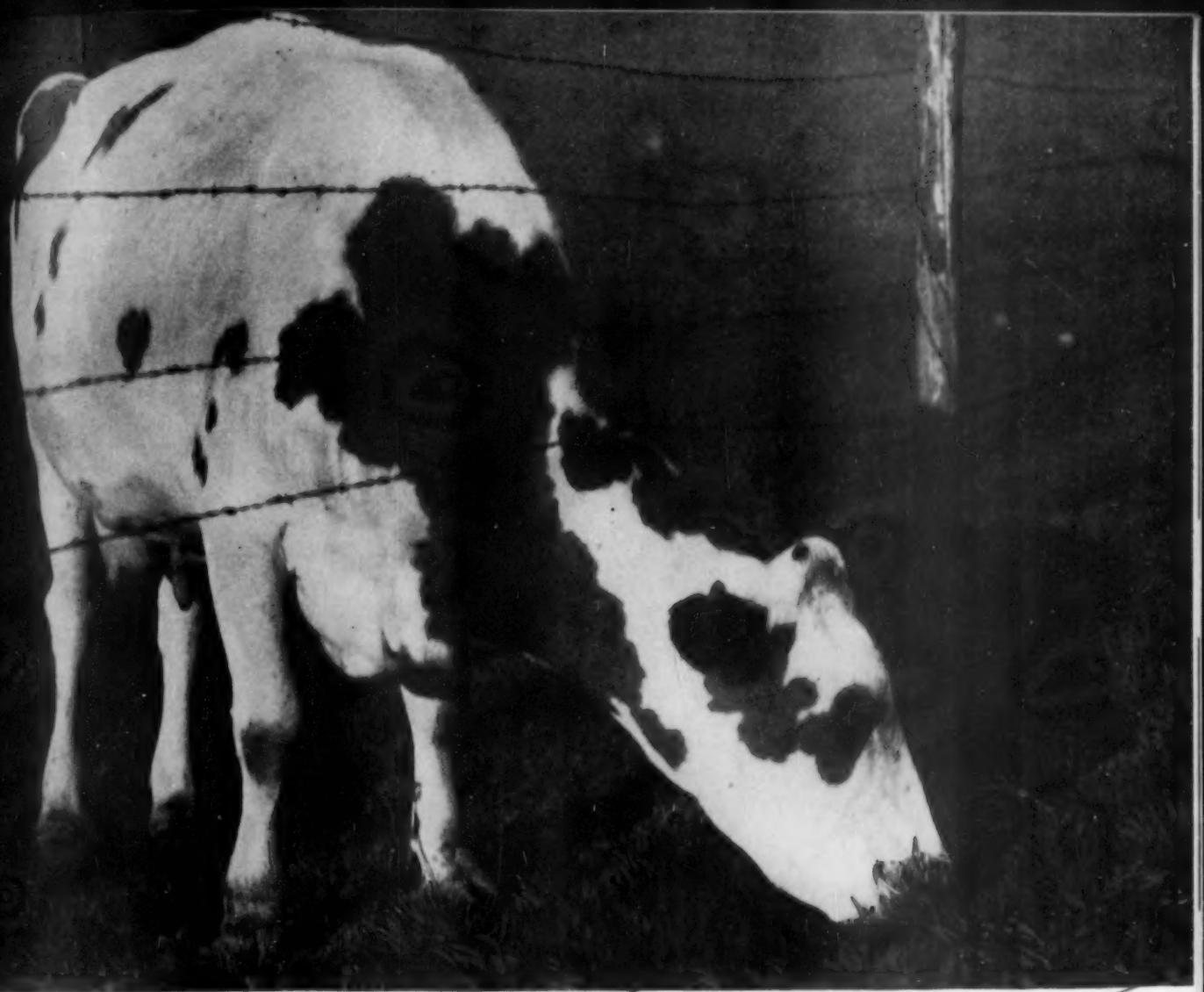
BRASS MAKES BULLETS

REGO Gas Plant Equipment to be safe, must be made of brass and other critical metals.

Conserve equipment—You may not get more.

The BASTIAN-BLESSING Co.
4241 W. PETERSON AVE. CHICAGO
Pioneers and Manufacturers of Precision Equipment for Using and Controlling High Pressure Gases

Is this YOUR Cleaning Problem?
Degreasing Steel Parts Thoroughly Before Welding
Whether you are using resistance, arc or gas welding, you'll find you can do MORE WORK . . . and do it EASIER and FASTER, too . . . if you insist FIRST on thoroughly cleaning steel plates, sheets, forgings, castings or parts.
Various Oakite alkaline materials and acid detergents are available which, by quickly, completely removing ALL oil, grease, drawing compound, heat scale or rust, provide CHEMICALLY CLEAN surfaces that help assure better, stronger, more uniform welds. Write today for helpful FREE data.
OAKITE PRODUCTS, INC., 18E Thames St., NEW YORK
Technical Service Representatives Conveniently Located in All Principal Cities of the United States and Canada
OAKITE
Tony Pounds for Victory
CLEANING



"THE GRASS IS *Always* GREENER . . !"

Need we finish it? . . . "on the other side of the fence."

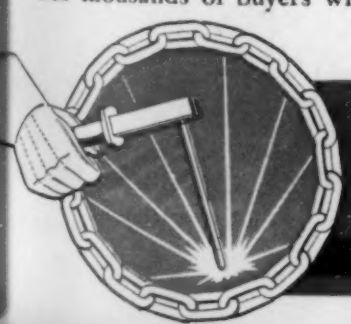
Here at The McKay Company we have no illusions about the very human tendency of most individuals to "shop around" a little in their quest for something better. And because of that deeply ingrained habit The McKay Company and the McKay line of products has grown in importance and reputation and usefulness.

For thousands of buyers who came to "nibble" have stayed to "graze" . . and other thousands, tempted elsewhere for awhile, have returned to the safe, dependable source of recurring satisfactory supply.

If it's stainless or mild steel Welding Electrodes or Commercial Chain you need (Tire Chains, too, although they may not be in your line), the reputation of these McKay products is sustained by 60 years of dependable service to industry

Try them sometime!

Sales Offices: York, Pa.



THE McKAY COMPANY
PITTSBURGH, PA.

WELDING ELECTRODES . . . COMMERCIAL CHAINS . . . TIRE CHAINS

Iron Works is the largest manufacturer of triple expansion engines for the Liberty Ship program in the country and have recently started the manufacture of complete driving units for the new Victory ships. These units consist of high-pressure steam turbines up to 8000 hp., and the necessary gear reducing units used in conjunction with the turbine drive.

They have plants at Sunnyvale, Long Beach, Ampere, N. J.—the latter being the Crocker-Wheeler Electric and Manufacturing Co., which they purchased outright in December last year.

Pomona Pump Co. is the oldest and largest manufacturer of turbine type pumps in the world, with headquarters at Pomona, Calif. and plants at Torrance, California and two plants in St. Louis, Mo.—one manufacturing Pomona pumps and one manufacturing Westco line of pumps which company was also taken over by Pomona. Needless to say the entire operations of the Joshua Hendy Iron Works organization is engaged 100% in the war program.

OBITUARY

Hans W. Roth

Information has reached us that our former member, Mr. Hans W. Roth, died on March 5, 1943, at Olten, Switzerland.

Mr. Roth was active in the introduction of resistance welding timing equipment, particularly of the mechanical type. Likewise, he developed the Multi-Hydromatic and Ultra-Speed types of resistance welding. He wrote many articles for the technical press and presented several interesting papers on timing and its importance in resistance welding.

E. P. Jeffery

E. P. Jeffery died on July 15, 1943, and is survived by a wife Margaret and daughter Jean.

Born in Detroit, May 3, 1887, son of Dr. and Mrs. Frederick Norwood Jeffery, Jeff's schooling ended abruptly when his father met reverses.

After a short period as a printer's devil, the modern miracles of the machine age, which started with the turn of the century, lured him from printing to producing.

He joined American Blower as a mechanic in 1906. With a rapidly growing company in an expanding industrial era, he advanced from mechanic to machine operator, to foreman of first, one department, then another and another, to assistant superintendent, and in 1925 he reached the rank of general superintendent. In 1930 he was made Works Manager, with supervision of all company plants and manufacturing.

In 1941, he was elected Vice-President of Manufacturing—the position he held so capably until his death.

In addition he was a director of the Canadian Sirocco Company, Past Chairman of Detroit Section of the A.W.S. and the Milwaukee Junction Manufacturers' Association.

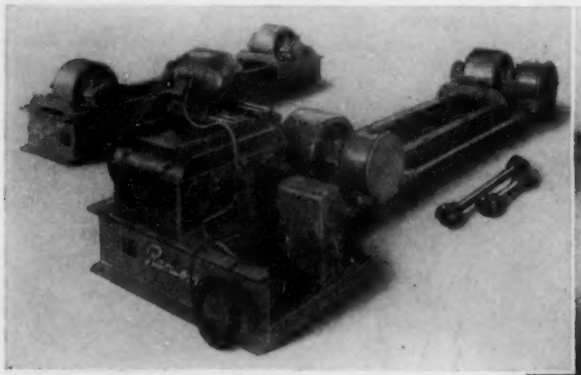
In all of his capacities he made friends and held them. He understood people because he loved people.

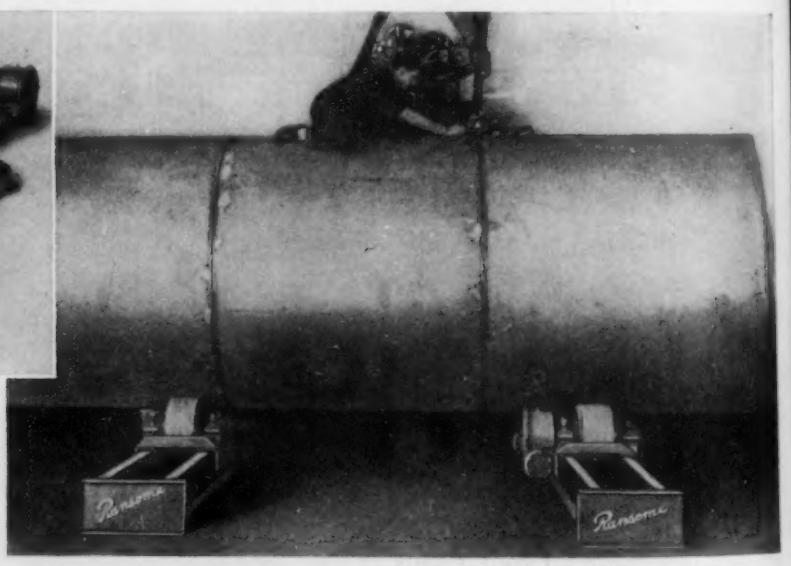
METALLIZING PROCESS AND EQUIPMENT CATALOG

The Metallizing Co., of America, 1330 West Congress St., Chicago, pioneer manufacturers of metal spray equipment, have just published a new 40-page catalog which deals with all phases of the Metallizing process. This new book is actually an encyclopedia on metallizing applications, with each page crammed with interesting, factual information on how this vital process is saving time, money and precious metals in production, maintenance and corrosion prevention.

All information contained in the book is classified according to industry and covers every major field in which metallizing is today effecting unbelievable economy in time and money in the maintenance and repair of costly and hard-to-replace equipment.

Another important piece of equipment developed by the Metallizing Co., and also described on the equipment pages, is the Mogul Link, a unit which simplifies arc welding. The Mogul Link produces a hi-cycle automatic current which serves as a "path" for the regular current produced by the welding machine. This "path" allows the welding current to





» TURNING ROLLS CAN STEP UP YOUR CYLINDER WELDING

For manual or automatic welding . . . here's the equipment that will enable your welders to produce at their top capacity. Smooth rotation at any desired speed . . . forward or reverse . . . is provided by a variable-speed drive with push-button control.

Work ranging from 2 to 14 feet diameter can be accommodated by shifting the position of the rollers.

Other Ransome Positioners include types and capacities for handling weldments of any size.

POSITIONING PAYS...WRITE FOR LITERATURE

Ransome WELDING POSITIONERS

INDUSTRIAL DIVISION • RANSOME MACHINERY COMPANY • DUNELLEN, NEW JERSEY

NEED SPEED?
TRY RACO TYPE D COMPOSITE ELECTRODES!

Where speed counts...and it always counts these days...give RACO Type D Composite Electrodes a trial!

A speed of 75 feet per hour on lap-welds on 3/16" plates can be attained with RACO Type D...not as a special stunt...but in regular production...without sacrificing strength or the soundness of the weld!

If you are working on mild steel...on range boilers and storage tanks ...galvanized material or sheet metal work such as pans and boxes...RACO Type D is the ONLY electrode readily available that will give results most closely comparable to standard shielded arc electrodes.

Will you make a test in your own plant? We will be glad to help you do it by sending samples of standard size RACO Type D Composite Electrodes. Just drop us a line telling the sizes you want for the test!

RACO SHIELDED ARC ELECTRODES

We are accepting orders for deferred delivery on:

RACO Type H-D No. 5 for deep groove welding in flat position AC or DC

RACO Type H-D No. 6 for fillets flat and positioned machine and plate construction AC or DC

RACO Type H-D No. 7 for all position, general construction — shipbuilding. DC Reverse Polarity

RACO Type H-D No. 8 for all position general purpose, single pass, building up wearing surfaces, low carbon. DC Straight Polarity or AC

Samples and Literature on Request

The REID-AVERY COMPANY

DUNDALK BALTIMORE 22 MARYLAND

flow much more smoothly and without interruption. It saves electrodes, saves current and improves welds. Regardless of the make or quality of arc welder used with the Mogul Link, the welder will weld with any welding rod, coated or bare, steel or alloy rods.

Perhaps one of the most interesting sections of the book is that which is devoted to questions and answers pertaining to Metallizing—questions which are most frequently asked about this miracle metal-spray process that transforms worn and discarded machine parts to "as good as new"—at a fraction of their original cost. All such questions are fully and explicitly answered in such a form that even the most inexperienced can understand.

Still another handy and informative section of the book is that which gives in easy-to-refer-to form an extensive list of Mogul Metallizing applications, thus at once indicating to the user of metallizing the many time and money-saving applications.

While the general offices of the Metallizing Co. are located at 1330 West Congress St., Chicago, there are, in addition, 21 divisional offices located in all sections of the country, with an office in Canada, and another in Honolulu, Hawaii.

Copies of the new Mogul Metallizing catalog are now available at no charge from the Chicago office of the company. Requests for catalogs should be written on company letterhead.

HOBART TRAINS WOMEN WELDERS

Women welders, after completing the welding course at the Hobart Trade School, Troy, Ohio, now are working side by side with men helping Uncle Sam get the production he needs in war industries. An increasing number of women are taking these welding courses to meet the shortage of male arc welders in war industries.

The Hobart Trade School is conducted by The Hobart Brothers Co., Troy, Ohio. Originally opened in 1941, the school has trained more than 4500 students in arc and acetylene welding. It is a non-profit organization established for the betterment of the welding profession.

The main course of study consists of 160 hr. of training, thoroughly covering the metallic arc welding of steel with plain and coated electrodes. For the students who have advanced through the main course, or for experienced steel welders, additional special courses by personal instruction are provided in welding non-ferrous metals or special alloys. The length of time required by students to become adept varies, of course, with each individual student, but the average period of training time is five weeks.

TRAINING SLIDEFILMS

By Lyne S. Metcalfe*

In a recent issue, THE WELDING JOURNAL published a review of the new series of training slidefilms, being widely used in the wartime industries, in pretraining welders.

* Any reader can get a print of this film for the nominal price of 65 cents by addressing The Jam Handy Organization, Detroit, Mich., producers for the Government.

● Everyone approves a bargain! And another way to interpret the attractive odds of 3 to 1 is—the installation of but *one* Sight Feed Acetylene Generator immediately brings you, Mr. Welder, *three* definite advantages:

1. **Assured** acetylene supply . . . independent of "bottlers"
2. **Visible** carbide supply . . . avoids large-job interruptions
3. **Economy** . . . saves YOU up to 75% in acetylene costs

So really you *don't gamble* when you select a Sight Feed Acetylene Generator. Your benefits are assured . . . conservatively stated, *one* will get you *three.*

THE SIGHT FEED GENERATOR COMPANY
Sales: Richmond, Ind. Factory: W. Alexandria, O.

Made of **AMPCO METAL** *—noted for High Physical Properties*

Aluminum Bronze Welding Rods Lick the "Tough" Jobs

—Wear-Resistant

You Can Select From 7 Grades

Ampco-Trode 10 All-purpose coated aluminum bronze electrode—for arc-brazing manganese bronze, brass, steel, or malleable iron. Higher strength and greater hot ductility than any other bronze weldrod.

Ampco-Trode 12 A soft, tough bronze with good bearing properties — Ampco Metal Grade 12 — for bronze overlays running against soft steel.

Ampco-Trode 16 A tough bronze of high tensile strength — Ampco Metal Grade 16—for bronze welds and overlays with good resistance to wear and impact.

Ampco-Trode 18 A strong bronze with good ductility—Ampco Metal Grade 18—unequalled for bronze overlays requiring the maximum bearing and wearing properties.

Ampco-Trode 20 A hard bronze—Ampco Metal Grade 20—for building up bearings working against hardened steel.

Ampco-Trode 21 A very hard bronze—Ampco Metal Grade 21—used for overlaying cast iron, steel or bronze for very hard bearings or on dies to reduce die wear and marking of the sheet metal.

Ampco-Trode 22 An extremely hard bronze—Ampco Metal Grade 22 — for overlaying drawing dies to avoid draw lines on the sheet metal being formed.

Gives Exceptional Service, Strength and Hardness

Behind the extraordinary ability of Ampco-Trode coated bronze electrodes to meet severe service conditions lie unequalled physical properties. Ampco-Trode is made from Ampco Metal, an alloy long recognized by America's Industry as superior for many services where ordinary bronzes fail.

With tensile strength up to 80,000 psi. and hardness to 352 Brinell, Ampco-Trode welding rods have greater strength and hardness than ordinary electrodes. Its excellent bearing characteristics make it desirable for applications such as building up worn shafts and wear strips or refacing bearings for extreme wear-resistance.

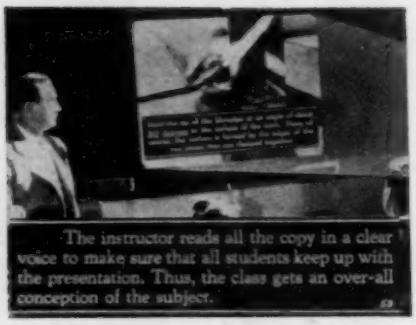
Ampco-Trode electrodes are extensively used by many U. S. arsenals, Navy yards, and Government contractors.

Made in 7 sizes—⅛" to ½" in diameter. Suitable for metallic arc, carbon arc, and oxy-acetylene welding.

Where strong tough welds are needed, try Ampco-Trode weldrods. With credit to yourself, you can solve that important problem of a suitable rod. Ask for free Ampco-Trode bulletin.

AMPCO METAL, INC.

DEPARTMENT WJ-9 MILWAUKEE 4, WISCONSIN



The instructor reads all the copy in a clear voice to make sure that all students keep up with the presentation. Thus, the class gets an over-all conception of the subject.

Inasmuch as the effectiveness of the picture screen medium depends, in large measure, upon the use of the proper teaching techniques, this series has now been supplemented with an 80-frame slidefilm titled, "Tips on Slidefilms" which gives pointers on the proper place and use of the medium in fundamental welding instruction. This picture is of the "reading" or discussional type, with letterings, legends, labels and notations to amplify the pictured techniques, and to guide the welding instructor in conducting the visual phase of teaching.

This slidefilm is available to any user of the picture screen, and is being widely used in the Armed Forces and in industry generally to improve the techniques of visual education.

The subject is divided roughly into five sequences, as follows:

1. Preparing the classroom.
2. The projector, and its use.
3. The projector in the classroom.
4. Individual use of the slidefilm by individual welding students as "refresher" material.
5. Use of slidefilms as a reference library in the workshop itself.

Tips on how best to plan and conduct a visual class in welding are given, and it is shown how the individual student may take a given scene from the film into the shop, project the pictures on wall, ceiling or floor and use them as a welding procedure "pattern."

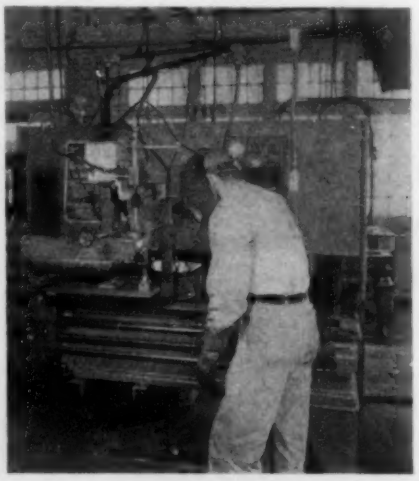
Typical of the pointers provided in this film are:

1. The meeting or classroom should be made ready before the group gathers.
2. The room should be reasonably dark, but no darkening is needed for small groups of 10 to 15 people.
3. The screen should be high enough to give all an unobstructed view of the pictures.
4. Never place the screen in a draft—a waving screen makes an out-of-focus image.
5. Place the projector just far enough from the screen so that the image will fill it.
6. Have the projector high enough so that the heads of those in the class will not obstruct the projected picture.
7. A small rug thrown over the projector cord will prevent anyone tripping over it.
8. Tie the projector cord to the table so that if someone should trip over it, the projector will not fall over.

Another sequence shows how the projector should be set up, and how to handle the film itself so as not to scratch the emulsion.

FABRICATION OF CYLINDERS

Atomic-hydrogen arc welding is used by the Chambers Corp., Shelbyville, Ind., to speed the fabrication of oxygen cylinders for the armed forces. For this work, General Electric 75-amp. atomic-hydrogen units furnish power to electronically controlled automatic welding equipment, while a G-E 35-amp. machine, for hand operation, is used for occasional touch-up welding and also to weld a bushing in the ends of the cylinders.



Operator Adjusting Control on Atomic-Hydrogen Arc-Welding Head Preparatory to the Start of Automatic Welding of Cylinder Seams; 75-Amp. Atomic-Hydrogen Power Source Can Be Seen to Front Right of Operator

These cylinders, which are made from sheets of alloy steel approximately 0.043 in. thick, are formed by being first rolled into tubes much like stove pipe. The butting edges are turned up in a slight burr and are then automatically fused together, without the use of filler rod, by being passed under the head of a 75-amp. atomic-hydrogen seam welder. This slight burr facilitates the welding process and improves the finished product.

Hemispherical ends are welded into the cylinders by the same 75-amp. machines. In this operation, once the ends are fastened in the cylinders the work automatically revolves under the welding head.

The finished cylinders are then tested under hydrostatic pressure up to 1000 psi., and checked for leaks with air under high pressure. Selected units are further checked by machine gun fire.

WELDED TENSILE TESTING MACHINE*

For Testing Welds in Both Flat Stock and Tubing in Giving Army-Navy Test to Welders at Piper Aircraft Corp., Lockhaven, Pa.

This tensile testing machine was designed and completely arc welded by Harry J. Shea, Division head of Tool & Die Dept., and Ralph C. Rathbun, Head of Welding Quality Control, Piper Aircraft Corp., Lockhaven, Pa.

In writing, Mr. Shea states that the design called for all parts to be electrically welded, saving approximately ten times the cost of castings and many weeks of delay in getting the machine in actual operation.

The pressure is furnished by four 8-ton hydraulic jacks all connected to one hydraulic pump located on the right

* This was one of the Prize Winning Entries in the Monthly Arc Welding News Contest conducted by The Hobart Brothers Co., manufacturers of Electric Arc Welders and Equipment.

Buy "Proven Fluxes" with Years of Guaranteed Satisfaction behind them

The Trade-Name is "ANTI-BORAX"

Ask for Them ***Unequalled for Quality***

A Flux for every metal: Cast Iron Welding Flux No. 1; Brazing Flux No. 2; Braz-Cast Flux No. 4, for bronze-welding cast iron; "ABC" Aluminum Flux No. 8 for sheet Aluminum and all alloys of Aluminum; Stainless Steel Flux No. 9; Silver Solder Brazing Flux No. 10; No. 16 Silver Solder Paste Flux.

ANTI-BORAX COMPOUND COMPANY
Fort Wayne, Indiana

SPEED UP WITH
SHAWINIGAN CARBIDE
SHAWINIGAN
FOR VICTORY BUY UNITED STATES WAR SAVINGS BONDS STAMPS
SHAWINIGAN PRODUCTS CORPORATION
EMPIRE STATE BLDG., NEW YORK

NATIONAL METAL CONGRESS *and* WAR CONFERENCE DISPLAY

"BASE OF OPERATIONS" for the Metal Industry

Chicago PALMER HOUSE **October 18 thru 22**

America's metal industry will move its "base of operations" to Chicago's Palmer House the week of October 18 to take full advantage of the 25th annual National Metal Congress and War Conference Display.

Devoted to increased production of war products, to conservation of materials, and to postwar planning, this year's Congress and Conference Display will bring vital new developments and ideas to the attention of thousands of war production executives and engineers in the metal industry.

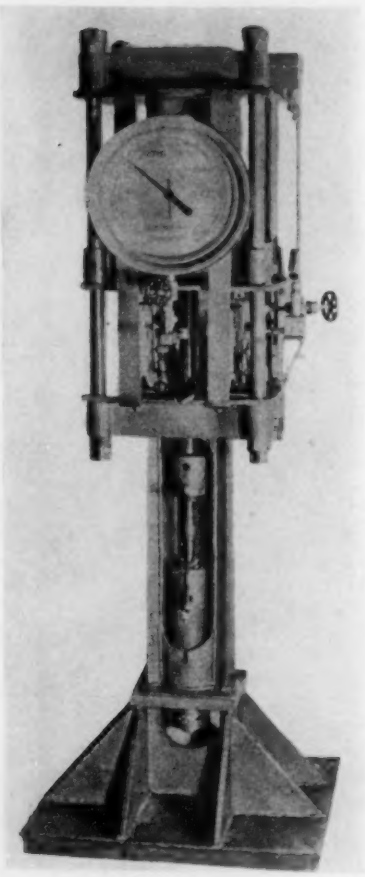
More than a hundred technical lectures will be presented by the four cooperating societies. Leaders in the metal industry and government will participate in the Special War Production and Conservation Sessions held each day by the American Society for Metals, sponsor of the event.

Hundreds of manufacturers will participate in the War Conference Display to be held in the Palmer House itself. These companies will have their engineers, their literature, light equipment and parts at these Conference Display rooms to help metal men solve their problems and increase production.

For complete information on these War Conference Displays — write or wire the American Society for Metals, 7301 Euclid Avenue, Cleveland 3, Ohio. Phone Endicott 1910.

NATIONAL METAL CONGRESS AND WAR CONFERENCE DISPLAY

Under the direction of the American Society for Metals, in cooperation with the American Welding Society, the Wire Association, and the Institute of Metals and Iron and Steel divisions of the American Institute of Mining and Metallurgical Engineers.



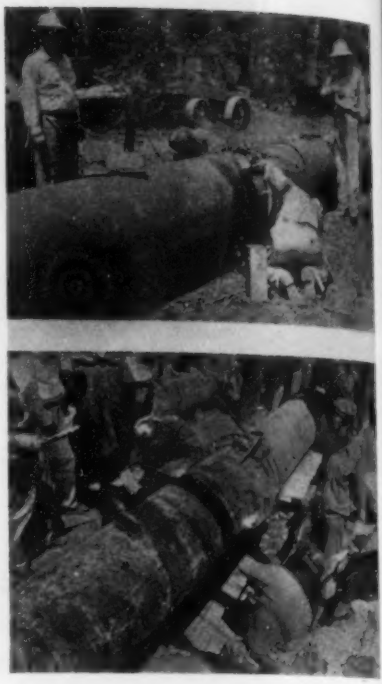
side of the machine. The hydraulic line is connected to the gage which registers pounds per square inch and also load pounds. A specimen can be seen in the illustration gripped in the two holding jaws just below the jacks.

A ball seat is located in the hydraulic line leading to the gage in order to hold the pressure until a reading of the test can be taken and also to relieve the sudden jar that occurs when the specimen breaks. The pressure can be relieved on the gage by opening a valve in a by-pass around the seat.

Besides being used for testing welds in giving the Army-Navy test to welders, the machine is also used for testing bends and shear tests by using the top plate above the upper screw.

This testing machine was constructed at a total cost of $500, including all labor and materials and has given very satisfactory service in constant use.

FINISHING "BIG INCH"



Courtesy The Lincoln Electric Co., Cleveland, Ohio

The first (upper photo) and the last (lower photo) welds of the 95 million dollar "Big Inch" pipe line which will deliver 300,000 barrels of oil a day to the oil-starved east coast. Oil was already flowing through pipes connected by the first weld when welders struck their arcs to make the final tie-in.

Arc welding, the process that has made possible so many miracles of war production, was responsible in no small way for the record breaking speed with which this line was constructed.

National

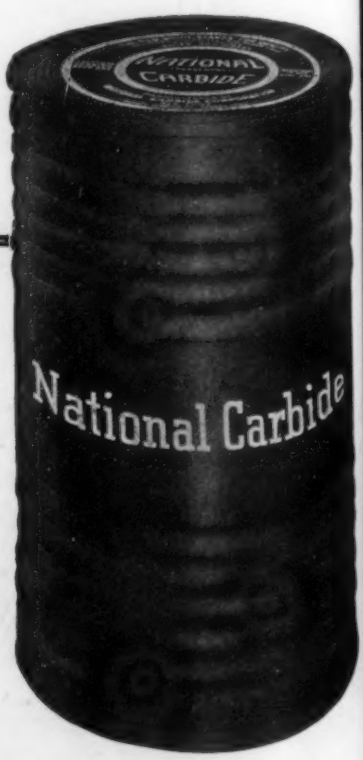
Carbide

IN THE RED DRUM

EFFICIENT

ECONOMICAL

DEPENDABLE



FOR WELDING and CUTTING

Use National Carbide in the Red Drum

60 E. 42nd St. NATIONAL CARBIDE CORPORATION New York, N. Y.

"They're a Sweet Pair for All-Around A. C. Work"

"With these two rods I can handle 'most any kind of mild steel welding when I want to use A.C.

"I use Alternex mainly with low open circuit, transformer type welders, and for light gauge work. It's colder than a Type A rod, so it's especially good where fit-up is poor.

"For heavy work with A.C., where you need deep penetration, good fusion and X-Ray perfect welds, I use the new Type A electrode. It's a honey for all-position welding, and especially in the overhead and vertical positions."

Send for the new Murex wall chart. It gives specifications on all Murex Electrodes, describes their applications and illustrates six ways of improving welding practice.

MUREX ELECTRODE	AWS-ASTM Class	IDENTIFICATION		Current	POLARITY		TENSILE STRENGTH OF DEPOSIT		ELONGATION OF DEPOSIT IN 2"	
		Color Of Coating	Color-Marking		Groove Welds	Fillet Welds	As Welded (p.s.i.)	Stress Relieved (p.s.i.)	As Welded %	Stress Relieved %
TYPE A	E-6011	Tan	End: None Secondary: Blue	A. C. or D. C.	Straight or Reverse	Straight or Reverse	62,000 to 71,000	60,000 to 68,000	22 to 26	27 to 32
ALTERNEX	E-6013	Pink	End: None Secondary: Brown	A. C. or D. C.	Straight	Straight	62,000 to 75,000	60,000 to 72,000	17 to 25	22 to 29

METAL & THERMIT CORPORATION

Specialists in welding for nearly 40 years. Manufacturers of Murex Electrodes for arc welding and of Thermit for repair and fabrication of heavy parts.

MUREX ELECTRODES

120 BROADWAY, NEW YORK

ALBANY • CHICAGO • PITTSBURGH
SO. SAN FRANCISCO • TORONTO

NEW PRODUCTS

The Society assumes no responsibility
for the validity of claims in this Section

"REGISTERED WELDER" STAMPS

To simplify compliance with governmental regulations requiring certification in connection with welds made on such products as aircraft, etc., New Method Steel Stamps, Inc., 147 Jos Campau St., Detroit 7, Mich. has announced a line of standard marking stamps for ready identification of welds made by certified operators.

The "registered welder" stamps developed by New Method are designed to simplify identification. In place of the conventional practice of using specially shaped symbols, the "registered welder" stamps comprise only a numeral or letter enclosed within a simple border such as a circle, an oval, a triangle or a square, etc.

The flexibility in design permits the assigning of individual stamps to every welder in a plant, no matter how many are employed. The "registered welder" stamps are also available in a variety of sizes—ranging from $^{5}/_{32}$ to $^{5}/_{16}$-in. borders, and including the usually popular $^{3}/_{16}$ and $^{1}/_{4}$-in. stamp sizes. Additional sizes are available on special order as are special symbols and borders.

IMPROVED ROCKER ARM ALUMINUM WELDER

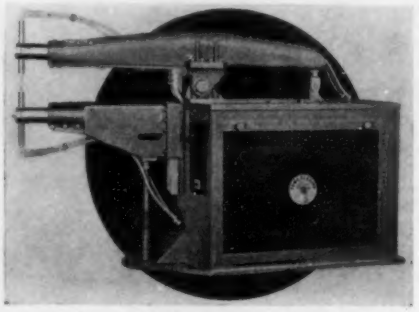
An improved rocker arm welder designed specifically for condenser-discharge welding of aluminum sheet has been announced by Progressive Welder Co., Detroit 12, Mich. Incorporating the most advanced features of design which have proved themselves as desirable as the result of experience to date, the new Model "B" is designed with exceptional rigidity and ruggedness.

Experience has indicated that there is a relationship between machine rigidity and frequency with which points have to be dressed.

Production experience with the new Model "B" indicates that the machine will require fewer point dressings in a day's run than other types now in use.

The new Rocker Arm Welder, according to Progressive, has passed all qualification tests and is already being used in the production welding of aircraft assemblies. It is designed to handle any combination of two thicknesses of 24-ST-Alclad, between 0.016 and 0.081 in. at over-all production rates of between 1000 and 2000 spots per hour—including point dressing time, etc.

Features of the machine include ad-

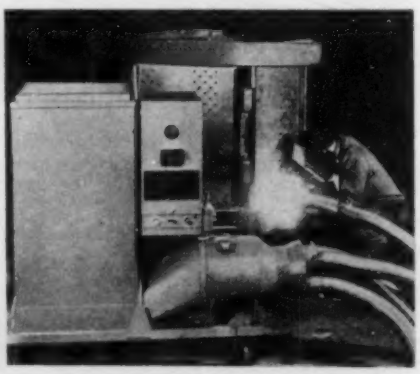


justable throat depth (from 36 to 42 in.) and a retractable upper arm to permit insertion of flanged work—this "retraction" being accomplished through a separate air cylinder controlled by an auxiliary foot switch.

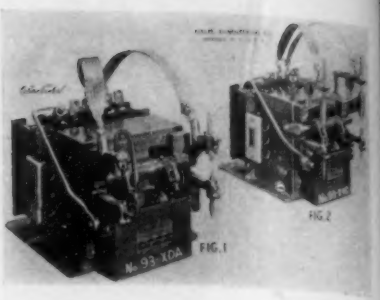
Both upper and lower arms are of the Universal type, and the lower knee with its arm is vertically adjustable over a range of 10 in. Welding arms are of $3^{1}/_{2}$-diameter hard drawn copper and current-conducting castings are of high-conductivity bronze to reduce voltage drop to a minimum.

ELECTRONIC WELDING FUME REMOVER

One of the latest uses of electronics in industrial functions is for removal of welding fumes directly from the scene of welding activity. A portable Westinghouse Precipitron electrostatic air cleaning unit



with three flexible metal hoses can be moved to any plant welding location. Welding fumes drawn through the hoses pass through the air cleaner where 90% of the fumes and air-borne particles are removed. The cleaned air is then returned to the room.



DOUBLE ELECTRODE SPOT WELDER

The illustration shows two types of a new Duplex Jaw Spot Welder recently introduced by the Eisler Engineering Co., Newark, N. J. These easily operated new bench type resistance welding machines are especially designed for efficient welding of light gage sheet and wire.

Each unit consists of two independent small welding machines built together with one transformer and a rheostat for finer heat adjustment. Figure 1 shows a model of a Duplex Welder equipped with rocker arm, Fig. 2, one supplied with vertical press arrangement. Both are foot operated by means of two treadles and chains and made in sizes from $^{1}/_{2}$ to 3 kva. The transformer used in both types has five secondary tap connections providing five different heat positions.

Step on foot treadle brings the operating jaws together squeezing the work by means of a lever system. On further pressing on treadle, contact is made and high welding current will flow from the secondary through the work until the switch automatically breaks the circuit. This complete welding operation takes but a fraction of a second. A skillful operator on this machine will make from 120 to 175 clean welds per minute.

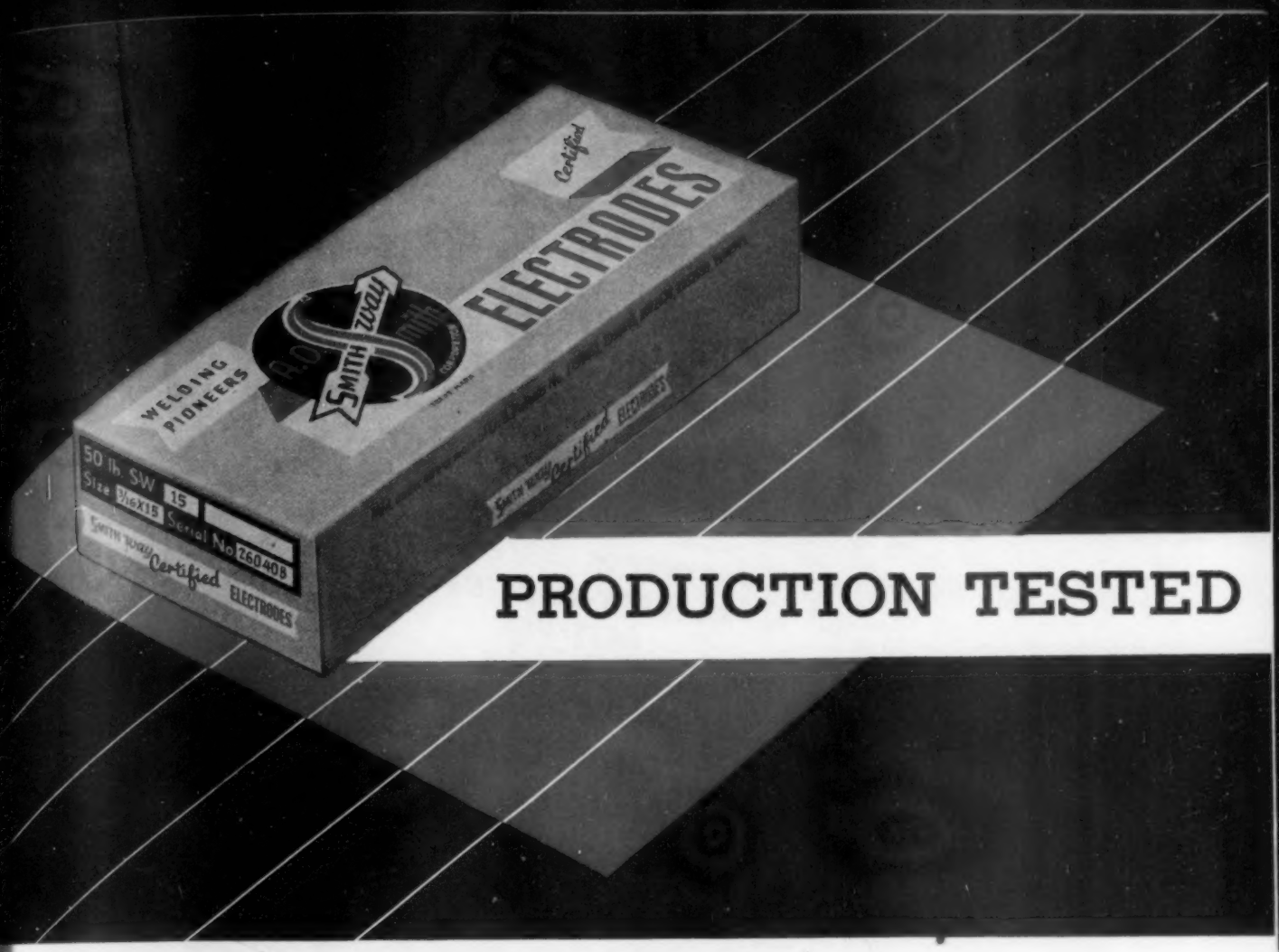
The machines are built for single-phase alternating current of 110, 220 or 440 volts, 25 to 40, 50 and 60 cycles.

The new Eisler Duplex Welders are production machines, especially qualified for welding of radio tubes, incandescent lamps, intricate optical and jewelry parts, metal novelties, etc.

AUTOMATIC SPOT WELDER

The new P-20 automatic press type spot welders were designed to meet present-day high-speed production requirements. Control of pressure, weld time, hold time, off time and speed up to 110 strokes per minute is entirely automatic. Unlike the manually foot operated welder—no effort by the operator is required. And unlike motor-driven welders which operate on a definite cycle, the new P-20 provides for variations in the cycle to suit each welding job.

The upper electrode is actuated by a double acting air cylinder and electrode pressure range can be adjusted by air pressure regulating valve. Current may be adjusted from 50% of maximum to 100% by the eight-point selector switch. Each of the two capacities made, the 20 kva. and the 30 kva., are available with throat



PRODUCTION TESTED

SMITHway Certified Welding Electrodes are products of laboratory and shop control unparalleled in the electrode manufacturing business. And that's important. No electrodes can be as good as SMITHway electrodes unless manufactured under equally strict quality control and unless hundreds of thousands of dollars have been invested in research.

But Smith goes even farther than that. SMITHway Electrodes are *production tested* in our own welding plants, which are among the largest in the world.

With us, the quality of electrodes is vital to the success of *everything we do!* To insure the quality, operating characteristics, and uniformity we *must* have, we manufacture our own electrodes. We also offer them to others who do welding. In so doing, we offer electrodes that have *proved* their ability to produce *better welding results* at *lower welding cost*— and proved it under actual shop conditions. We think SMITHway Certified Welding Electrodes can do as much in your plants.

Mild Steel . . . High Tensile . . . and Stainless Steel
WELDING ELECTRODES
made by welders . . . for welders

SMITHway Arc Welder eliminates objectionable arc blow; speeds production.

SMITHway Welding Monitor trains better welders faster.

A.O. SMITH *Corporation*

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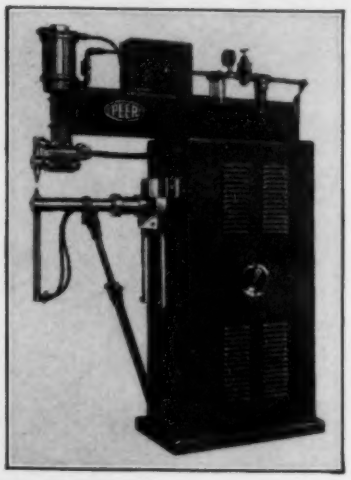
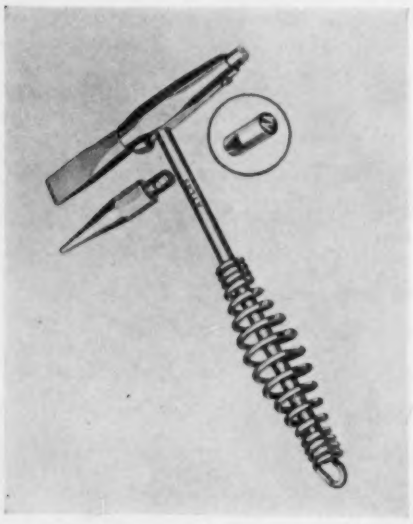


depths of either 12 or 24 in. Priced at under $1000 the P-20 welder quickly pays for itself through greater production and uniformity of welding done. These welders are made by the Pier Equipment Manufacturing Co., Benton Harbor, Mich.

WELD SYMBOL HAMMER

A combination weld marking tool and chipping hammer known as the Atlas Symbol Hammer has been introduced by Atlas Welding Accessories Co., 307 Boulevard Building, Detroit 2, Mich., manufacturer of weld cleaning tools.



This tool features a keyed symbol in the form of alphabetical and numerical combinations or distinctive outline symbols which are available in combinations to place permanent and definite responsibility for every weld and to identify individual welders in crews up to 15,000. This large number of combinations make it possible to include inspectors' marks in the identification system.

Marking tips are interchangeable; likewise, bits are replaceable at chipping end. Symbol and bit shanks are positioned and held in a snug socket-fit by set screws to prevent loosening. Weight of the Atlas Symbol Hammer is 16 oz.; length is $10^1/_2$".

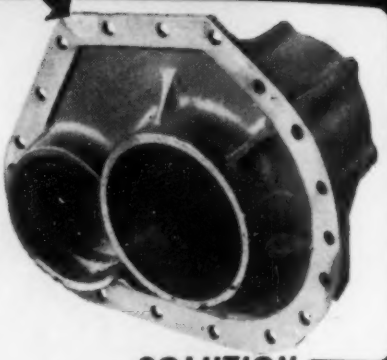
Castolin Eutectic®

EUTECTIC

(Means Lowest Binding Alloy)

LOW TEMPERATURE WELDING

SALVAGING DEFECTIVE ALREADY-MACHINED CASTINGS WITH MACHINABLE EUTECTIC "COLDARC" ALLOY NO. 24

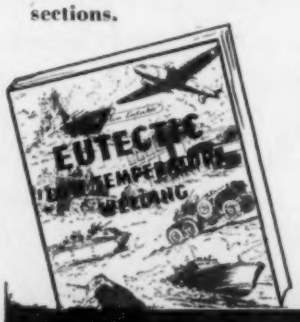


PROBLEM

Salvaging already-machined cast-iron, malleable iron or chrome-nickel alloy steel castings having porous spots or machining errors. . . . When ordinary electrodes were used a hardened fusion line generally resulted. Naturally, this caused great difficulty and time-waste in machining the welded sections.

SOLUTION

Eutectic "Coldarc" Alloy No. 24 (AC-DC). This new kind of electrode can be applied at low heat—without preheating—without deep fusion of the parent metal. Eutectic "Coldarc" Alloy No. 24 gives perfectly *color matching, easily machinable* welds on all ferrous metals. It has been saving hundreds of thousands of dollars in foundries, tool shops, ordnance plants, etc.



Only "Eutectic® Alloys" are the true Low Temperature Welding Alloys that are revolutionizing production welding, maintenance welding and salvaging in war plants throughout the nation. There are 42 specialized rods for every metal and every welding job. Developed and manufactured only by Eutectic Welding Alloys Company.

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Quality

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Super-flexibility and long life make "MAJOR" the preferred brand welding cable. Copper conductor consists of fine bare wires, properly stranded to give extreme flexibility.

A separator is applied over the conductor to strip rubber in order to leave bare copper for quick, clean attaching to lugs and electrode holders.

Rubber jacket is WPB approved, extruded type cured by continuous vulcanization process, which produces a tough, durable and waterproof cover. This cable offers maximum resistance to abrasion and it is high in tensile strength.

SIZES AVAILABLE
No. 6 to No. 4/0
75 to 550 Amps.
PROMPT SHIPMENTS

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MILWAUKEE, WISCONSIN
MANUFACTURERS—DISTRIBUTORS
OFFICES AND AGENTS ALL OVER THE UNITED STATES

SECTION ACTIVITIES

CHATTANOOGA

The Chattanooga Section held their annual meeting on July 2nd at the Hotel Patten. The following officers were installed at that time: *Chairman*, Pembroke O. Leach; *Vice- Chairman*, Justin Brown; *Secretary-Treasurer*, R. G. Wilson.

Also installed at the same time were Roy Lorentz as *Program Chairman* and Clarence Miller as *Membership Chairman*. Members appointed to serve as the Executive Committee for the coming year are as follows: R. B. Bayston, R. P. Smith, R. M. Daniels, A. J. Moses, E. C. Chapman.

After dinner and the election and installation of officers, Mr. A. J. Moses of the Combustion Engineering Company delivered an informative talk on the "Welding of Wood" based on a lecture by Professor McPherson of San Diego, Calif. Mr. Moses' talk was highlighted by a series of charts and diagrams prepared by Mr. Roy Lorentz of Combustion Engineering Company.

Mr. E. C. Chapman, Southern District Vice-President, has been appointed a member of the Post-War Planning Committee which has been formed by the various engineering societies of Chattanooga.

The next regularly scheduled meeting of the Chattanooga Section will be in September.

CHICAGO

On July 15th the members of the Chicago Section entertained their wives and lady friends at a dinner and dance at the Edgewater Beach Hotel on Chicago's North Side.

This was the first so-called "Ladies Night" to be held by the Chicago Section. There were 57 guests and apparently all had an enjoyable time. After a dinner in the Marine Dining Room the guests took tables on the Beach Walk while they enjoyed a floor show and an evening of dancing.

Such a good time was had by all that plans are now going forward to make this an annual event of the Chicago Section.

INDIANA

At a meeting of the executive committee of the officers-elect for the Indiana Section, the following were appointed: J. R. Wirt, *Chairman*, R. H. Imes, *Vice-Chairman*, George Mann, *Secretary*, Members Executive Committee, E. F. Gilyeat, retiring Chairman, and P. B. Hall, P. Grubbs, C. M. Sauer, E. O. Smith, C. W. Mercer, R. D. Eaglesfield, Sr.

KANSAS CITY

At a directors' meeting held on August 4th the activities of the Kansas City Section for the coming year were discussed.

It was decided to continue holding meetings in the Pine Room of Fred Harvey's located in the Union Station, on the second Tuesday of each month, instead of the first Tuesday as heretofore. Meetings will start in September and continue through May, with the exception of the month of October, which meeting will be dispensed with because of a large number of members attending the Annual Meeting in Chicago at that time.

MILWAUKEE

The following officers and directors have been elected by the Milwaukee Section for the ensuing year: *Chairman*, E. C. Brekelbaum, Harnischfeger Corp.; *Vice-Chairman*, C. Malmberg, Allis Chalmers Mfg. Co.; *Secretary-Treasurer*, G. F. Meyer, Machinery & Welder Corp. *Directors:* Keith Walker, C. Hart, H. Pollei, R. Walbridge, C. Schoenbaum, R. Mueller, C. Swift.

At the meeting of the Fifth Annual Informal Get-together and Election of Officers, a dinner party was held and Mr. K. L. Hansen, president of the Society, was the honored guest, and favored those present with a short talk after which a short colored motion picture was shown, and the balance of the evening was spent in dancing and entertainment. The meeting was held at the Pfister Hotel.

MOBILE

The Mobile Section inaugural dinner was held at the Admiral Semmes Hotel on July 8, 1943, with well over 200 members and guests in attendance. After a thoroughly enjoyable dinner, Mr. James F. Lincoln, Jr., of The Lincoln Electric Company of Cleveland, Ohio, delivered a paper on "Welding Rod Selection and the Value of Positioning." Mr. Lincoln's statements were supported by slides with the result that everyone enjoyed them and was benefited.

Since this was their first meeting, the Section was extremely fortunate in being able to secure Mr. Lincoln as principal speaker.

Among the guests were Dr. Taylor of the University of Alabama Extension System from Tuscaloosa, Alabama, and Mr. Paul W. Brunson of the University of Alabama Center, Mobile, Alabama.

NORTHERN NEW JERSEY

The following officers and executive committee were elected by the Northern New Jersey Section for 1943–44: *Chairman*, R. E. Powell, Western Electric Co.; *Vice-Chairman*, J. George Blind, Midwest Piping & Supply Co.; *Secretary-Treasurer*, F. G. Flocke, The International Nickel Co. *Executive Committee:* R. E. Powell, J. George Blind, F. G. Flocke, J. H. Bucknam, F. C. Fyke, W. H. Hart, K. W. Horsman, Alexander Kidd, P. F. McEvoy,

MALLORY STANDARDIZED WELDING ELECTRODES

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LESS DOWN TIME
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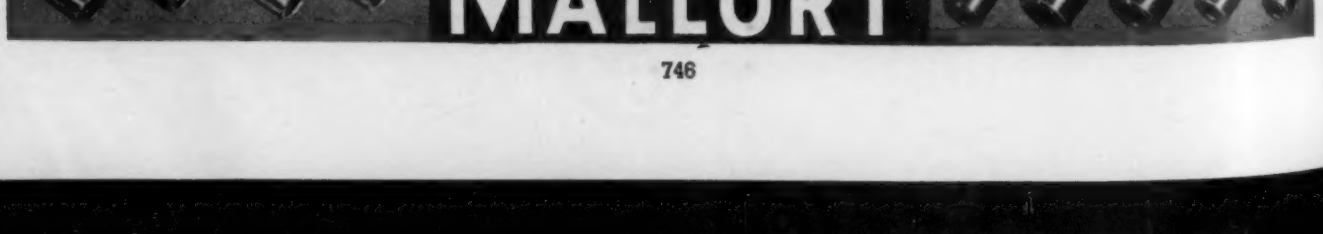

M. W. Mitchell, G. W. Nigh, T. B. Smith, F. B. Swankhous.

OKLAHOMA CITY

The following have been elected as officers and executive committee of the Oklahoma City Section for the 1943–44 season: *Chairman*, M. G. Wicker; *Vice-Chairman*, H. N. Simms; *Secretary*, J. W. Blake; *Treasurer*, J. R. Black. *Executive Committee:* James M. Craig, M. L. Mossholder, Wm. T. Tiffin, B. O. Underwood.

SAN FRANCISCO

The June meeting of the San Francisco Section was held at the Engineers Club, San Francisco, Calif., on Friday, June 25, 1943. There were approximately 250 members and guests in attendance at this meeting. This was an open discussion meeting, and many problems and new ideas were brought up for discussion.

The New General Electric picture, "Atomic Hydrogen Welding," was shown along with slides showing the various applications to which atomic hydrogen welding can be advantageously applied —with comments from Mr. E. W. Olds, Field Service Engineer of the Victor Equipment Co.

SAVANNAH

Mr. J. E. Durstine of The Lincoln Electric Company put on a program using polarized light, polariscope and project image models on a screen to demonstrate various stresses and strains of welded joints, at the July 8th meeting of the Savannah Section. At this meeting Mr. F. M. Gilbert was elected to succeed Mr. J. R. Cowart, Secretary-Treasurer, who resigned because of ill health.

Employment Service Bulletin

POSITION VACANT

V-127. Wanted: Chief Inspector. College graduate, metallurgical and welding experience preferred—by long-established manufacturer of welding electrodes and other welding products. Permanent position. Give full details in first letter.

List of New Members

July 1 to July 31, 1943

ATLANTA

Coons, Phillip H. (C), Warner Robins, Ga.
Gordon, Morris L. (C), P.O. Box 263, Warner Robins, Ga.
Merritt, John W. (C), 125 Colquitt St., Macon, Ga.
Norwood, William O. (C), Warner Robins, Ga.
Turner, Herman H. (C), 2349 Second St., Macon, Ga.
Woods, Oscar T. (C), Iron Kettle Inn, Byron, Ga.

BOSTON

Des Jardins, Ovila J. (C), 132 Willow Ave., Somerville, Mass.
Fadden, Peter V. (C), 19 Dartmouth St., Lynn, Mass.
Herres, S. A. (C), Watertown Arsenal, Watertown, Mass.
Tobin, Richard S. (C), 75 Pitts St., Boston, Mass.
Watters, Elmer R. (D), 617 Dwelly St., Fall River, Mass.
Wheeler, Thomas (B), S. D. Hicks Engrg. Co., 1671 Hyde Park Ave., Hyde Park, Mass.
Wilson, George F. (C), 93 Aster Circle, Weymouth, Mass.

CHICAGO

Ahalt, Frank W. (C), 4638 Middaugh Ave., Downers Grove, Ill.
Fosha, Henry K. (C), 723 Bluff St., Waukegan, Ill.
Herr, Clarence C. (C), 501 N. Central Ave., Chicago, Ill.
Kullen, Bruno B. (C), Crane Co. Met. Lab., 4100 Kedzie Ave., Chicago, Ill.
McHale, J. E. (B), 2220 W. 37th St., Chicago, Ill.
Phillips, Wesley E. (C), 112 N. Ashland Ave., La Grange, Ill.
Singer, Morse B. (C), Machine Tool Lab., University of Illinois, Urbana, Ill.
Sobotka, John, Jr. (D), 3745 So. Westley Ave., Berwyn, Ill.
Towne, Miles A. (C), 817 South Home Ave., Park Ridge, Ill.
Wegrzyn, Edward L. (B), American Manganese Steel Div., 389 E. 14th St., Chicago Heights, Ill.

CINCINNATI

Blighton, William (C), 371 Baum St., Cincinnati, Ohio.

CLEVELAND

Bartlett, Kenneth M. (B), Thompson Products, Inc., 2196 Clarkwood Rd., Cleveland, Ohio.
Moore, Howard H. (C), 4508 West Ave., Ashtabula, Ohio.
Schutz, Carl T. (D), 119 Arizona Ave., Lorain, Ohio.

DETROIT

Sinner, Albert (B), 231 East Troy, Ferndale, Mich.

INDIANA

Miller, C. J. (C), 1816 N. Pennsylvania Ave., Indianapolis, Ind.

KANSAS CITY

Chisholm, P. J. (B), Keystone Trailer & Equipment Co., 1501 Guinotte St., Kansas City, Mo.
Kramer, A. A. (B), Columbian Steel Tank Co., 1509 West 12th St., Kansas City, Mo.

LOS ANGELES

Famighetti, Ted (D), 1104–13 St., San Pedro, Calif.
Mehuson, J. G. (C), 2443 East Slauson Ave., Huntington Park, Calif.
Remmerde, John C. (C), Route 1, Box 519, Upland, Calif.
Senft, Edison (D), 512 South Chester Ave., Compton, Calif.
Spalding, Luther P. (B), 510 South Flower St., Inglewood, Calif.

LOUISVILLE

Anderson, Ivan C. (B), R.R. 2, Georgetown, Ind.
Dettmar, William (A), The American Air Filter Co., Inc., 215 Central Ave., Louisville, Ky.
Fritsch, R. E. (A), Tube-Turns, Inc., 224 E. Broadway, Louisville, Ky.
Hawkins, Jesse N. (C), 3316 Lexington Road, Louisville, Ky.
Jones, Raymond Edward (D), 1620 South Second St., Louisville, Ky.
Kretzer, Arthur A. (B), R.R. 4, Box 739, Louisville, Ky.
Lee, Art (D), Route 2, Box 202, Jeffersonville, Ind.
Lewis, Theodore (A), Jeffersonville Boat & Mach. Co., Jeffersonville, Ind.
Liva, Aurelio (B), 1116 Euclid Ave., Louisville, Ky.
Lytikainen, Fred (B), 116 South 35th St., Louisville, Ky.
McGrew, Wilbur H. (D), 318 South 41st St., Louisville, Ky.
McMonigle, George (B), Mauckport, Ind.
Ray, Joseph C. (A) Louisville Bridge & Iron Co., 11 & Oak Sts., Louisville, Ky.
Rolli, J. W. (B), Republic Welding Co., 305 E. College St., Louisville, Ky.
Stonecipher, Oscar J. (B), 4628 Brewster Ave., Louisville 11, Ky.
Webendorfer, Chester (C), 403 South Columbus Ave., Mt. Vernon, N. Y.
Windle, Edwin F. (D), 835 East Washington St., Louisville, Ky.
Wright, Louis W. (C), 2628 Virginia Ave., Louisville, Ky.

MARYLAND

Wilhide, Glen C. (B), The Black & Decker Mfg. Co., Towson, Md.

MILWAUKEE

Gerhardt, J. T. (C), Theo Kupfer Fdy.-Iron Wks., 101–149 Waubesa St., Madison, Wis.
Steiner, Arthur F. (C), 7747 Port Washington Rd., Sta. F, Route 9, Milwaukee, Wis.

MOBILE

Beldon, William Frank (B), 35 Lancaster Rd., Springhill, Ala.
Elgin, Frank S. (B), 323-A Dunlap Circle, Chickasaw, Ala.
Kelly, William E. (C), 1561 Luling St., Mobile, Ala.
Martin, Julius H. (B), P.O. Box 509, Chickasaw, Ala.
Weldon, William F. (B), P.O. Box 249, Mobile, Ala.

NEW YORK

D'Espies, Arthur (C), Welding Alloys Mfg. Co., 744 Broad St., Newark, N. J.
Feller, Milton S. (D), 1184—43rd St., Brooklyn, N. Y.
Johnson, Roy (B), Arenco Machine Co., 25 West 43rd St., New York, N. Y.
Klein, Irving (B), United Metal Box Co., Inc., 174 7th St., Brooklyn, N. Y.
Stein, Sam (D), 470 Atkins Ave., Brooklyn, N. Y.
Sullivan, Cornelius J. (C), 34–13 101 St., Corona, L. I., N. Y.
Valyo, John A. (B), Bitumen Welding Supply Co., 202 Banker St., Brooklyn, N. Y.
West, Walter (D), 161 Eckford St., Brooklyn, N. Y.

NORTHERN NEW JERSEY

Abbott, William R. (B), Lock Joint Pipe Co., 150 Rutledge Ave., East Orange, N. J.
Chelborg, Rudolph (B), Linde Air Products Co., 686 Frelinghuysen Ave., Newark, N. J.
Coryell, Cornelius R., Jr. (B), Lock Joint Pipe Co., 150 Rutledge Ave., East Orange, N. J.
Dowden, George B. (D), R.C.A. Victor Div. of Radio Corp., 415 South 5th St., Harrison, N. J.
Hubbard, R. A. (B), Westinghouse Electric & Mfg. Co., Div. 318, Bloomfield, N. J.
La Malfa, Anthony (B), Lock Joint Pipe Co., 150 Rutledge Ave., East Orange, N. J.
Linden, John H. (D)., 3304 Palisade Ave., Union City, N. J.
Miller, Edwin V. (B), Box 238, Aruba Netherland West Indies.
Timmerman, Paul E. (B), Linde Air Products, 686 Frelinghuysen Ave., Newark, N. J.

NORTHERN NEW YORK

Westbrook, Jack H. (F), R.F.D. 3, Pinewoods Ave. Rd., Troy, N. Y.

NORTHWEST

Olson, Harold E. (B), 5618 Blaisdell Ave. South, Minneapolis, Minn.

PHILADELPHIA

Charlton, E. J. (B), Lukenweld, Inc., Coatesville, Pa.
Decker, Frank (D), 332 N. 2nd St., Camden 2, N. J.
Dooley, Charles Lewis (B), 1224 Potter St., Chester, Pa.
McCue, Walter W. (D), 307 Orlando Ave., Gloucester, N. J.
O'Mara, Michael (C), 1529 Spring Garden St., Philadelphia 30, Pa.

PUGET SOUND

Cellers, Montgomery Q. (B), Rte. 3, Box 556, Seattle, Wash.
Marker, M. C. (D), 111 Naval Ave., Bremerton, Wash.
Patnik, Albert (C), 1914 Utah Ave., Seattle 4, Wash.

ROCHESTER

Clark, Wayland B. (C), 2001 Ridge Road West, Rochester 13, N. Y.
Duemmel, Frank R. (C), 962 Glide St., Rochester 6, N. Y.
Kazorski, Walter V. (C), 217 Bluff Drive, E., Rochester, N. Y.

SAN FRANCISCO

Borchert, Carl (B), Kaiser Cargo, Inc., Shipyard No. 4, Richmond, Calif.
Hamre, John A. (C) 541 Maine St., Vallejo, Calif.
Hodgetts, P. J., Cpl. ASN, 39163538 (C), A.P.O. 957 c/o Postmaster, San Francisco, Calif.
Ladd, George W. (B), 2036 W. Harding Way, Stockton, Calif.
Maaske, Hal (B), Kaiser Cargo, Inc., Shipyard No. 4, Richmond, Calif.
Page, Dave (B), The Permanente Metals Corp., Shipyard No. 2, P.O. Box 1072, Richmond, Calif.
Randall, Merle (B), Stuart Oxygen Co. Research Lab., 2295 San Pablo Ave., Berkeley 2, Calif.
Sisk, Chas. C. (D) 837 Geary St., Apt. 606, San Francisco 9, Calif.
Soukup, Phil (B), The Permanente Metals Corp., Shipyard No. 2, P.O. Box 1072, Richmond, Calif.
Sundin, Melvin A. S.F., 2/C (D), U.S.S. *Holland*, c/o Postmaster, San Francisco, Calif.

SAVANNAH

Alexander, C. E. (B), 902 Kehon Court, Savannah, Ga.
Badger, Otis Clarke (B), 21 4th St., Tattnall Homes, Savannah, Ga.
Boyd, William Edward (B), 223 E. Gwinnett St., Savannah, Ga.
Brantley, C.B. (B), 44 Kayton Court, Tattnall Homes, Savannah, Ga.
Collins, Lloyd (C), 315½ Tattnall St., Savannah, Ga.
East, L. M. (B), 543 Francis Bartow Place, Savannah, Ga.
Farthing, E. E. (B), 1417 E. 39th St., Savannah, Ga.
Forsythe, Dale (B), 70 Crescent Court, Savannah, Ga.
Goithe, James S. (B), 118 W. Harris St., Savannah, Ga.
Hughes, L. H. (B), 220 West Henry St., Savannah, Ga.
Kelly, M. L. (B), 4 W. Oglethorpe St., Savannah, Ga.
Kimel, H. R. (B), 610 W. 37th St., Savannah, Ga.
Meadows, A. F. (B), 609 H. Court Rowland, Savannah, Ga.
Parr, John Carrol (B), 4 West Oglethorpe St., Savannah, Ga.
Perry, J. M. (B), 420 E. Broughton St., Savannah, Ga.
Seitz, Ray Campbell (B), 610 W. 37th St., Savannah, Ga.
Shaffer, Douglas E. (B), 311 E. Liberty St., Savannah, Ga.
Smith, J. F. (B), 1732 E. Gwinnett St., Savannah, Ga.
Stanley, G. E. (B), 319 W. President St., Savannah, Ga.
Tuten, Manasse H. (B), Rte. 3, Harrock Hall, Savannah, Ga.
Way, Roger L. (B), 116 E. Duffy St., Savannah, Ga.

TOLEDO

Baldwin, Edward L. (B), Rte. 1, Box 308 AA, East Toledo, Ohio.
Conti, Sam (B), 224 Lincoln Dr., Port Clinton, Ohio.
Harwick, Berton Leo (C), 613½ Buffalo St., Toledo, Ohio.
Hockenberry, James D. (B), 347 Raymer Blvd., Toledo, Ohio.
Hutchison, Emerson (B), Rte. 1, Lasalle, Mich.
Kingsbury, Linford (B), Box 158, Deerfield, Mich.
Massenberg, J. W. (B), 3217 Island Ave., Toledo, Oh o.
Micek, Michael S. (C), 315 East Park St., Toledo, Oh o.
Perry, Paul B. (C), Rte. 1, Lasalle, Mich.
Simmers, Harry E. (C), 1216 Camden St., Toledo, Ohio.
Wellman, Carl O. (C), 1502 Erie St., Toledo, Ohio.

WASHINGTON

Goldberg, Taft (C), 913 Kent Rd., Apt. 2, Richmond, Va.
Ricketts, Arthur G. (C), P.O. Box 1598, Norfolk, Va.
Shadonix, John H. (C), 9 Dale Dr., Portsmouth, Va.

WESTERN NEW YORK

Elliott, Frank F. (C), 123 Sheffield Ave., Buffalo, N. Y.
Jenkins, E. S. (B), Curtiss-Wright Corp., Research Laboratory, Buffalo, N. Y.

WICHITA

Overby, Paul (D), R.R. 1, Box 753, Wichita, Kans.

NOT IN SECTIONS

Collins, Louis, S.Sgt., ASN20141482 (C), Battery A, 237th, AAA, S.L. BN., A.P.O. 928, c/o Postmaster, San Francisco, Calif.
Curls, Aaron Barnard (C), Food Machinery Corp., Lakeland, Fla.
Davis, Harold G. (B), P.O. Box 4272, Portland 8, Ore.
Hirschmann, Franz (C), International Welding Co. Ltd., 2, South Parade, London, W.4, England.
Lawton, C. C., Lt. Colonel, QMC. (C), Headquarters, QMRTC, Camp Lee, Va.
Lindholm, John (B), R.D. 1, Gordonville, Pa.
Persey, Elbert W. (C), Rte. 10, Box 605, Milwaukie, Ore.
Thatcher, C. R. (B), 101 Pavilion Way, Field End Rd., Eastcote, Middlesex, England.

TENTATIVE PROGRAM

TWENTY-FOURTH ANNUAL MEETING AMERICAN WELDING SOCIETY WEEK OF OCTOBER 18, 1943

HOTEL MORRISON, CHICAGO, ILLINOIS

Make Room Reservations Early—Direct with Hotel Morrison
Arrive Sunday 17th—(Convention Hotel) Rates Apply for Short or Long Stay

IMPORTANT

ALL Technical Sessions will positively start on time.

No Stenotype Reporter—Members desiring to discuss papers are urgently requested to prepare discussion in writing in advance of the meeting and to send copies to headquarters as those preparing written discussion will be given preference at the sessions. Members and guests giving extemporaneous discussion at meeting should forward a written transcript of discussion as soon as possible after the meeting.

Members and guests are urged to REGISTER immediately upon arrival at A.W.S. HEADQUARTERS and obtain Convention Badge admitting them to Exposition.

NO REGISTRATION FEE but a charge of $1.00 (per member) and $1.50 (non-member) is made for Technical Sessions Papers.

Registration hours throughout the week:

Hotel Morrison

Monday, Tuesday and Wednesday 9 A.M. to 5 P.M.
Thursday 9 A.M. to 12 Noon

MONDAY, OCTOBER 18th, Morning—9:30 A.M.

OPENING SESSION

Mural Room

Chairman—K. L. HANSEN,
President, American Welding Society

Vice-Chairman—H. V. DAVID,
Chairman, Convention Committee

PRESENTATION OF MEDALS AND PRIZES

The Adams Lecture, by Dr. C. A. ADAMS, First President, American Welding Society

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Monday Afternoon—2:00 P.M.

SIMULTANEOUS TECHNICAL SESSIONS

CUTTING

Roosevelt Room

Chairman—ELLSWORTH L. MILLS,
Bastian-Blessing Co.
Vice-Chairman—H. E. ROCKEFELLER,
The Linde Air Products Co.

Flame Cutting Heavy Sections and Large Diameters
by R. L. DEILY and E. BENYO, Bethlehem Steel Company

Rolling Mill Maintenance and Production Welding Problems
by T. W. MORGAN, American Rolling Mill Co.

Electronic Control of Gas-Cutting Machines
by R. D. McCOMB, General Electric Co.

Machine Cutting for Assembly Line Fabrication
by CHARLES O. ADAMS, Delco Products

RAILROAD

Rooms 329-331

Chairman—A. G. OEHLER,
Editor, Railway Age
Vice-Chairman—C. E. MORGAN,
Chicago, Milwaukee & St. Paul R. R.

The Welded Locomotive Boiler
by E. G. YOUNG, University of Illinois

Welding as It Applies to Railroads
by ROBERT MORAN, Missouri Pacific R.R.

Locomotive Parts Fabrication and Welding of Bronze Bearing Surfaces
by J. W. KENEFIC, Air Reduction Sales Co.

Welding of Railroad Transport Equipment
by C. DREUTZLER, Electro-Motive Corp.

WELDABILITY

Mural Room

Chairman—A. B. KINZEL,
Union Carbide & Carbon Res. Labs.
Vice-Chairman—C. H. JENNINGS,
Westinghouse Elec. & Mfg. Co.

A System for the Preservation of Ductility in Weldments
by GILBERT E. DOAN and ROBERT D. STOUT, Lehigh University

Cooling Rates Associated with Arc Welding and Their Application to the Selection of Welding Conditions
by WENDELL F. HESS, L. L. MERRILL, E. F. NIPPES, JR., and A. P. BUNK, Rensselaer Polytechnic Institute

The Weldability of Silicon Steels
by C. E. JACKSON and G. G. LUTHER, Naval Research Laboratory

Weld Bead Hardness Tests on Some Carbon, Nickel and Nickel-Chromium War Department Steels
by OSCAR E. HARDER and C. B. VOLDRICH, Battelle Memorial Institute

Establishment of Cooling Curves of Welds by Means of Electrical Analogy
by VICTOR PASCHKIS, Columbia University

Monday Evening—7:30 P.M.

EDUCATIONAL SESSION

Mural Room

Chairman—E. R. SEABLOOM, Crane Company

Vice-Chairman—R. E. McFARLAND, Western Electric Co.

General Electric Film on Atomic Hydrogen Welding

An Address on Welding in Ordnance Construction, by COL. S. B. RITCHIE, Chief of the Service Branch of the Office of the Chief of Ordnance.

TUESDAY, OCTOBER 19th, Morning—9:30 A.M.

SIMULTANEOUS TECHNICAL SESSIONS

RESISTANCE WELDING

Roosevelt Room

Chairman—A. F. DAVIS, The Lincoln Electric Co.

Vice-Chairman—C. L. PFEIFFER, Western Electric Co.

Flash Welding of Alloy Steels
by C. M. MANZER, General Electric Co.

The Spot Welding of Heavy Gauges of Carbon and Alloy Steels
by JOHN C. BARRETT, Taylor-Winfield Corp.

The Spot Welding of the Carbon Steels SAE 1020, SAE 1035 and SAE 1045 in the 0.040-In. Thickness
by WENDELL F. HESS and D. C. HERRSCHAFT, Rensselaer Polytechnic Institute

TUBING

Rooms 329-331

Chairman—J. P. DODS, Summerill Tubing Co.

Vice-Chairman—H. S. CARD, Formed Steel Tube Institute

Electric Resistance Welded Steel Tubing—Its Quality Control and Application
by R. D. MALM, Clayton Mark & Co.

Machine Welded Metal Tubing
by G. C. GRIDLEY, Mechanics Universal Joint Div., Borg-Warner Corp.

Gas Weld and Furnace Weld Tubing for Construction Purposes
by A. C. WEBER, Laclede Steel Co.

FLAME HARDENING AND HARD FACING

Embassy Room

Chairman—D. H. COREY, The Detroit Edison Co.

Vice-Chairman—R. G. HAWLEY, Link Belt Ordnance Co.

Ordnance Flame Hardening
by STEPHEN SMITH, Air Reduction Sales Co.

Hard Facing in the War on Wear
by J. A. GALLAHER, Haynes Stellite Co.

Reclamation by Hard Facing
by F. G. JONES, Wall-Colmonoy Corp.

SHIPS

Mural Room

Chairman—DAVID ARNOTT American Bureau of Shipping

Vice-Chairman—F. M. SCOTTEN, Great Lakes Engineering Works

Welding in Ship Construction
by H. A. MATIS and A. M. UNGER, Pullman-Standard Car Mfg. Co.

Prefabrication of Welded Ships in a Structural Steel Plant
by E. T. BLIX and J. C. ARNTZEN, Mississippi Valley Structural Steel Co.

The Control of Welding in Ship-building
by R. W. BRENDLE, Tampa Shipbuilding Co.

Tuesday Afternoon—2:00 P.M.

SIMULTANEOUS TECHNICAL SESSIONS

RESISTANCE WELDING

Roosevelt Room

Chairman—R. E. POWELL, Western Electric Co.

Vice-Chairman—B. L. WISE, Federal Machine and Welder Co.

Why the Weld Recorder
by J. R. FETCHER and J. VAN DEN BEEMT, E. G. Budd Mfg. Co.

Double Pressure Systems as Applied to Resistance Welding Machines
by S. M. HUMPHREY, Taylor-Winfield Corp.

Stored Energy Welding of Mild Steel
by J. M. DIEBOLD, Yellow Truck and Coach Mfg. Co.

Flash Welding of Nickel and High Nickel Alloy Rod
by W. F. HESS and A. MULLER, Rensselaer Polytechnic Institute

SHIPS

Rooms 329-331

Chairman—JAMES C. BLAKE, Commander, USNR, Bureau of Ships

Vice-Chairman—H. W. PIERCE, New York Shipbuilding Corp.

Planning for Production Welding and Cutting in a Modern Shipyard
by W. B. BOWEN, Ingalls Shipbuilding Corp.

Thermit Welding in Maritime Commission Work
by J. H. DEPPELER, Metal and Thermit Corp.

Longitudinal Welded Joints
by A. G. BISSELL, Bureau of Ships, Navy Dept.

RESEARCH

Mural Room

Chairman—H. C. BOARDMAN, Chicago Bridge & Iron Co.

Vice-Chairman—T. S. FULLER, General Electric Co.

Fatigue Strength of Fillet Welded Joints
by W. M. WILSON, University of Illinois

Discussion of Means for Evaluating Weldability of Alloy Steels
by 1st Lt. S. A. HERRES, Watertown Arsenal

The Effect of Normalizing on the Properties of Welds in Carbon-Molybdenum Steel Pipe
by I. A. ROHRIG, D. H. COREY and SABIN CROCKER, The Detroit Edison Company

Correlation of Structure and Strength of Spot Welds in Aluminum Alloys
by DANA SMITH and F. KELLER, Aluminum Company of America

Evening—7:30 P.M.

University Research Conference—H. C. BOARDMAN, Chairman, Presiding

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WEDNESDAY, OCTOBER 20th, Morning—9:30 A.M.
SIMULTANEOUS TECHNICAL SESSIONS

AIRCRAFT
Roosevelt Room

Chairman—G. N. SIEGER, S-M-S Corporation
Vice-Chairman—J. R. DAWSON, Union Carbide & Carbon Res. Labs.

Fatigue Strength of Welded Aircraft Joints
by T. V. BUCKWALTER, The Timken Roller Bearing Co.

The Spotweldability of Low Carbon and Other Aircraft Steels
by L. C. BIBBER and JULIUS HEUSCHKEL, Carnegie-Illinois Steel Corp.

Weldability Tests of Aircraft Structural Steels
by C. B. VOLDRICH and R. D. WILLIAMS, Battelle Memorial Institute

NON-FERROUS WELDING AND BRAZING
Rooms 329-331

Chairman—R. L. KOHLBRY, Machinery and Welder Corp.
Vice-Chairman—E. E. ISGREN, R. G. LeTourneau, Inc.

Modern Welding Methods for Copper and Copper Alloys
by J. J. VREELAND, Chase Brass and Copper Co. Inc.

Carbon Arc Welding of Naval Brass
by KENNETH L. WALKER, Foster Wheeler Corp.

Aluminum Brazing Sheet—Fundamentals of Metal Flow
by M. A. MILLER, Aluminum Research Laboratories

PIPING
Mural Room

Chairman—F. C. FANTZ, Midwest Piping and Supply Co.
Vice-Chairman—F. C. FYKE, Standard Oil Development Co.

Oxyacetylene Distribution Systems
by D. F. GUTHRIE and R. W. STEWART, Air Reduction Sales Co.

Field Welding of Steel Penstocks and Tunnel Linings
by L. G. CHRISTOFFERSON, Chicago Bridge & Iron Co.

Weldability of 27% Chrome Steel Pipe
by R. A. MUELLER, I. H. CARLSON and E. R. SEABLOOM, Crane Co.

Wednesday Afternoon—2:00 P.M.

SIMULTANEOUS TECHNICAL SESSIONS

AIRCRAFT
Roosevelt Room

Chairman—G. S. MIKHALAPOV, War Metallurgy Committee, National Research Council
Vice-Chairman—R. T. GILLETTE, General Electric Co.

The Heliarc Welding of Magnesium Alloys as Used in Airplane Fabrication and Substitution for Critical Materials
by T. E. PIPER, Northrop Aircraft, Inc.

Fatigue Studies of Welded Test Triangular Structures with NE8630 Steels
by A. J. WILLIAMSON, Summerill Tubing Co.

Fabrication and Welding of Aircraft Exhaust Tubing - Manifolds and Headers
by C. D. LaFOND, Buhl Stamping Co.

Metallic Arc Welded X4130 Steel Tubing
by W. T. TIFFIN, University of Oklahoma

INSPECTION, QUALIFICATION AND TRAINING
Mural Room

Chairman—O. B. J. FRASER, International Nickel Co.
Vice-Chairman—L. M. SKIDMORE, General Motors Institute

Symposium on Applicable Methods of Inspection of Arc Welding for
(1) **Shipbuilding**—J. LYELL WILSON
(2) **Structural**—R. B. LINCOLN
(3) **Pressure Vessels and Piping**—W. D. HALSEY
(4) **Aircraft**—J. B. JOHNSON
(5) **Machinery**—N. L. MOCHEL

Welding Operator Training
by A. N. KUGLER, Air Reduction Sales Co.

REPAIR AND MAINTENANCE
Rooms 329-331

Chairman—L. S. McPHEE, Whiting Corp.
Vice-Chairman—E. C. BRECKELBAUM, Harnischfeger Corp.

Bronze Welding in the Design and Maintenance of Forming Dies
by C. E. SWIFT, Ampco Metal, Inc.

The Job Shop—Past, Present and Future
by F. W. SHACKLETON, Central Ohio Welding Co.

Maintenance of Tools and Equipment
by T. B. JEFFERSON, The Welding Engineer

Reclamation of Perishable Tools by Low Temperature Brazing
by W. A. JOHNSON, International Harvester Co.

Evening—6:30 P.M.

Section Officers Dinner and Conference

THURSDAY, OCTOBER 21st, Morning—9:30 A.M.
SIMULTANEOUS TECHNICAL SESSIONS

AIRCRAFT
Roosevelt Room

Chairman—P. H. MERRIMAN, Glenn L. Martin Co.
Vice-Chairman—H. O. KLINKE, Republic Aviation Corp.

Quality Control of Aircraft Spot Welding
by NATHAN C. CLARK, Lockheed Aircraft Research Lab.

The Application of Flash Welding in Airplane Construction
by C. B. SMITH, Douglas Airplane Co.

Spot Weld Joint Efficiency for Aluminum Alloys
by C. W. STEWARD, Curtiss-Wright Corp. Research Laboratory

Spot Welding of Magnesium Alloys
by W. S. LOOSE, Aircraft Welding Research Committee WRC.

PRODUCTION
Rooms 329-331

Chairman—A. M. CANDY, Hollup Corp.
Vice-Chairman—OTIS L. SMITH, Weldit Acetylene Co.

Automatic Arc Welding Solves Production Problems
by R. F. WYER, General Electric Co.

Transition from Riveted to Welded Construction of Light Combat Tanks
by WM. C. OSHA and J. W. SHEFFER, American Car and Foundry Co.

Welding of Armored Tanks
by WILLIAM BOESE, Pullman Standard Car Mfg. Co.

PRESSURE VESSELS
Mural Room

Chairman—ISAAC HARTER, Babcock & Wilcox Co.
Vice-Chairman—W. E. CRAWFORD, A. O. Smith Corp.

War Emergency Codes for Welded Pressure Vessels
by E. R. FISH, Hartford Steam Boiler Insp. & Ins. Co.

The Hammer Test for Welded Pressure Vessels: An Investigation
by C. O. DOHRENWEND, Armour Research Foundation

Thursday Afternoon—2:00 P.M.

Business Meeting
Roosevelt Room

3:00 P.M.

Board of Directors Meeting

SOCIETY HEADQUARTERS—HOTEL MORRISON, CHICAGO

In order not to be disappointed, members of the Society are urged to make reservations promptly by writing to Leonard Hicks, Jr., Morrison Hotel, Chicago. It is imperative that these letters mention the name "American Welding Society Convention."

The rates for single rooms are $3.30, $3.85, $4.40, $5.50. The rates for double rooms are $4.40, $4.95, $5.50, $6.60, $7.70, $8.80. Rooms with twin beds are from $5.50 up. A large number of suites are available ranging in price from $9.90 on up depending upon the number of rooms and space.

Inasmuch as it is believed that the Hotel will be filled, members are requested to indicate their exact stay so that the rooms may be rented to members coming in later.